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FREEWAY CAPACITY ANALYSIS WITH MICROSCOPIC SIMULATION MODEL (FRESIM)

PHASE I - FINAL REPORT

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CHAPTER 1

INTRODUCTION

This report has been prepared by the Institute of Transportation at The New Jersey Institute of Technology, for the Federal Highway Administration (FHWA). The major purpose of this report is to identify deficiencies and limitations, and recommend suggestions in using the microscopic freeway simulation model, FRESIM, for approximation of freeway capacity.

1.1 Project Description

The Highway Capacity Manual (HCM) and several traffic analysis computer packages (i.e., HCS, FREFLO, and FREQ, etc.) are being challenged with the problem of dealing with freeway capacity as input information. However, applying a microscopic simulation model such as FRESIM (FREeway SIMulator) to the problem, freeway capacity can be observed through a series of runs. This type of testing can detect, and address important issues that arise when applying FRESIM, which include the following: (1) What are the factors affecting freeway capacity? (2) How sensitive are the identified factors affecting freeway capacity? (3) What are the deficiencies in FRESIM which should be modified and/or enhanced in order to determine freeway capacity accurately? (4) What are the general guidelines to determine freeway capacities under various conditions? The relationship between freeway capacity and the application of FRESIM will be addressed, and examples will be given.

The major thrust of the effort in this study will concentrate on identifying the factors affecting freeway capacity, indicating deficiencies, and recommending potential improvements in FRESIM for analyzing freeway capacity. The capacity of simple freeway configurations, such as pipelines, single isolated lane drops, single isolated ramp merges, single isolated lane diverges, and a weaving section containing an on-ramp and an off-ramp; also, more complicated freeway networks, will be analyzed under a systematic methodology after the suggested improvements have been completed.

1.2 Objectives and Scope

Understanding the capacity of a freeway section is critical to the evaluation of its level of service and the development of congestion control measures. Although FRESIM is one of the best freeway microscopic simulation programs available [37], it does have some limitations and deficiencies for analyzing freeway capacity. The objectives of this project are:

(1) To address deficiencies of FRESIM, which may generate some biased results.
(2) To recommend modifications to FRESIM, which would improve the result accuracy.
(3) To develop an appropriate methodology, which can approximate freeway capacity.

1.3 Report Organization

Phase I of the report is organized into five chapters. Chapter 1 is the introduction of the project, which describes the background, scope, and objectives of the project. Chapter 2 presents an overview of the microscopic freeway simulation model (FRESIM) and previous studies in freeway capacity analysis. Chapter 3 identifies deficiencies and limitations in the FRESIM, while Chapter 4 provides recommendations for the deficiencies identified in Chapter 3. Finally, a brief summary of this project is stated in Chapter 5.

CHAPTER 2

REVIEW OF MICROSCOPIC FREEWAY SIMULATION MODELS

A thorough review has been conducted of literature, current practices, and relevant software (e.g., FRESIM source code) for analyzing freeway capacity. The findings of the review included materials directly relevant to the factors affecting freeway capacity in the microscopic freeway simulation model FRESIM. This review is organized and described in three sections, including: (1) Historical Background of the FRESIM Model, (2) Characteristics of the FRESIM Model, (3) Previous Studies Related to Driver Population, (4) Previous Studies Related to Freeway Capacity Analysis, and (5) Other Related Studies.

2.1 Historical Background of the FRESIM Model

The microscopic freeway simulation model, FRESIM, is an enhanced and reprogrammed version of the INTRAS (Integrated Transportation Simulation) model. The INTRAS model was developed for the FHWA in the late 1970s to access the effectiveness of freeway control and management strategies (e.g., ramp metering and incident detection) [43], and was operational on a mainframe computer.

As a part of FHWA's TRAF group of traffic simulation programs, the FRESIM model was developed by the JFT associates in 1990. Unlike INTRAS, FRESIM runs on a microcomputer, and it is a microscopic and stochastic freeway traffic simulation program. In simulating freeway traffic, it considers traffic and geometric conditions, driver characteristics, and vehicle performance. Modeling vehicle movement, such as car-following, lane changing, and crash avoidance; maneuvers are programmed within the FRESIM Model. Over the years, FRESIM has been extensively enhanced, tested, and validated [4].

2.2 Characteristics of the FRESIM Model

FRESIM is a microscopic freeway simulation model. It treats each vehicle as a separate entity and models traffic based on their complex moving maneuvers [6], such as car-following, lane-changing and crash. Individual vehicle attributes are selected from an embedded vehicle operational characteristics table. It can simulate entire freeway networks including frontage roads, urban arterial streets, meters, and detectors. It can evaluate incident detection algorithms, real time ramp metering strategies, traffic operations, geometric design strategies, and weaving sections.

FRESIM can handle various geometric configurations of freeways; which include (1) variation in grades, radius of curvature, and super-elevations on the freeway, (2) lane additions and drops anywhere on the freeway, (3) freeway blockages, (4) work zones, (5) up to three lane ramps and inter-freeway connectors, and (6) up-to five through lane freeway mainlines, with up to three auxiliary lanes. FRESIM can simulate various operational features such as pre-timed and traffic responsive ramp metering controls, comprehensive representation of the freeway surveillance systems, heavy vehicle movements, and up to sixteen different vehicle types, ten driver types, and the reaction of drivers to upcoming geometric changes.

2.3 Previous Studies Related to Driver Population

Only two articles were found related to "driver population factor." Both of the articles were written by Sharma. One article [40] was based on his other article, "Road Classification According to Driver Population" [41].

Some earlier studies attempted to evaluate the impact of non-local drivers or non-commuters on freeway traffic capacity. We found that not much research specifically quantified the magnitude of the driver population factor. Lu, Huang, and Mierzejewski [23] concluded that driver population adjustment factor (f_p), ranging between 0.75 and 1.00 in the HCM [17], was determined based on the traffic studies performed by Caltrans in the early 1970s.

More recently, Sharma [40, 41, 42] showed considerable interest in analyzing the driver population factor. His primary contribution was classifying roadways in terms of traffic composition. He classified roads by the percentage of urban commuters and highly recreational trips. The driver population factor for urban commuter traffic would be 1.0 and for highly recreational traffic would be 0.75. As shown in Table 2-1, he identified five classes of driver types, and associated a different driver population factor with each. Master traffic patterns of seasonal, daily, and hourly traffic variations were built in his study to categorize roadway traffic streams in some sites in Alberta, Canada. His study considered daily and hourly volume patterns, and also examined the traffic variations between weekday and Sunday volumes.

Table 2-1: Recommended Population Adjustment Factors by Sharma

Traffic Stream Types	Population Adjustment Factors
Urban Commuters	1.00
Regional Commuters	0.95
Regional Recreational and Commuters	0.90
Inter-regional	0.85
Long Distance	0.85
Long Distance and Recreational	0.80
Highly Recreational	0.75

Sharma's study used traffic characteristics to determine driver population. This method could be called the indirect measurement, because it utilized indicators to reflect driver populations rather than trying to determine driver population directly from traffic flows. The direct measurement method is more practical and cost effective compared with traditional direct measurements, such as roadside interviews, and license plate surveys. The specific techniques used in Sharma's study were too complicated for general practitioners, and the assignment of f_p values was purely judgment based; albeit logical.

2.4 Previous Studies Related to Freeway Capacity Analysis

Previous studies that deal with freeway capacity and/or other various factors, such as speeds, affecting freeway capacity, are reviewed and summarized below. All the findings in the previous studies may assist us in identifying the deficiencies and recommending improvements for the FRESIM model.

Literature related to freeway speed studies [10, 11,15, 19, 32] mainly deal with the posted speed limit on a freeway segment. Most studies [10, 11, 15, 19] found that the speed limit influenced vehicle speed variations, the average speed, and the 85^{th} percentile speed.

Speed variation has an impact on freeway capacity, especially on freeways with a less number of lanes, because the existence of slow vehicles may substantially reduce freeway capacity[4].

A study was conducted by Jernigan and Lynn [19] on the impact of speed limits on freeways. According to their study for Virginia's rural interstate highways, they indicated that the average speed was 56.3 miles per hour (mph), and the 85th percentile speed was 62 mph, when the posted speed limit was 55 mph. One year later, the posted speed limit was increased to 65 mph, the average speed had increased to 63.5 mph and the 85th percentile speed to 70 mph. In another study, Pfefer, Stenzel, and Lee [32] found that an increase of 4 mph occurred in the 85th percentile speeds of vehicles along rural interstate highways in Illinois, after the speed limit was changed from 55 mph to 65 mph.

In 1992, Freedman and Williams [10] found that the mean free flow speed on rural interstate highways with a 55 mph speed limit was 63 mph; exceeding the speed limit by 8 mph. However, on rural interstate highways with 65 mph speed limit, the mean speed was approximately 67 mph; exceeding the speed limit by only 2 mph. Differences between the 85th percentile speed and the speed limit was also greater on highways with a 55 mph speed limit, than that of highways with a 65 mph speed limit. Therefore, if the posted speed limit increases, the speed variation might decrease.

A study conducted by Harkey et. al. [15] on the assessment of current speed zoning criteria, found the following:

- the mean speed exceeded the posted speed limit by 1 to 8 mph
- the 85th percentile speed ranged from 6 to 14 mph over the posted speed limit or 4 to 7 mph over the mean speed, and
- the majority (70.2 percent) of free-flow drivers did not comply with posted speed limits.

Garber and Gadiraju [11] conducted a study on identifying factors affecting speed variance. That research concluded: (1) The difference between the design speed and the posted speed limit had a statistically significant effect on the speed variance. (2) The minimum speed variance was achieved if the posted speed limit was between 5 and 10 mph lower than the design speed. (3) Drivers had a tendency of driving at increasing speeds as the roadway geometry improved, regardless of the posted speed limit.

Nemeth and Rathi [29] conducted a simulation study by using FRESIM and indicated the potential impact of speed reduction at freeway lane closures. They found that compliance with the reduced speed limit had no significant impact on the number of uncomfortable decelerations, but it reduced variance in speed distribution. Their results did not support the assumption that the effective speed reduction at work zones would create a potentially hazardous disturbance in traffic flow.

Speed characteristics near freeway lane closure areas were studied by Rouphail and Tiwari [39]. They found that the speed distribution upstream of a work zone followed a normal distribution when no queuing condition existed. In a lane closure area, the speed distribution showed significant variance regardless of the quality of upstream flow of the lane closure.

A study on "Performance Differences on Driving and Laboratory Tasks Between Drivers of Different Ages" was conducted by Ranney and Pulling [36]. They gave lower performance ratings to older drivers than younger drivers. The lower rating reflected a number of differences, including more time in decision making, error in route selection, gap execution, vehicle control, and difficulty in understanding task instructions. No difference between the two groups were observed in driver response to emergency situations.

An investigation of the effect of change in vehicular characteristics on highway capacity was conducted by Radwan and Kalevela [35]. Their results showed that despite the

change in vehicular characteristics, there was no discernible corresponding change in highway capacity parameters.

Michaels and Fazio [26] developed a model for vehicle merging on freeways based on local driver behaviors. Their model assumed that a ramp accepted a gap based on the first order motion vectors of an approaching vehicle, which was perceptually the angular velocity.

Blumenfeld and Weiss [2] developed a model for merging vehicles from an acceleration lane of finite length based on an assumption that the main stream speed was greater than that on the acceleration lane. The acceleration speed was constant up to the acceleration lane, and then there was instantaneous stop. Gap acceptance function was assumed to be a step function and the distribution of vehicle headways was assumed to be negative exponential. Their findings suggested that the expected delays did not depend strongly on the length of the acceleration lane, and that most of the merging occurred near the beginning of the acceleration lane. Increasing the length of the acceleration lane beyond a certain length resulted in relatively little benefit in merging.

Pahl [33] used arterial data to determine the average accepted gap and the lag gap in gap-acceptance maneuvers for vehicles close to their exiting off-ramps and for through-vehicles based on a function of distance from the vehicle to the off-ramp. He found that the size of the accepted gap decreased as the vehicle approached its exiting ramp. The average gap size accepted by exiting and through vehicles decreased as traffic flow increased. He also found that a larger accident risk was expected by an exiting vehicle than by a through vehicle in changing lanes. Whereas the accident risk imposed on the lag car of the accepted gap was smaller for lane changes performed by an exiting vehicle, than by a through vehicle.

In another study, Pahl [34] empirically determined lane-change frequencies of exiting vehicles close to their intended off-ramps and of through vehicles on four, six and eight

lane freeways. He found that the greatest frequency of exiting vehicle lane changes was toward the right lane and that a corresponding increased frequency of through vehicle lane change toward the left lane, and as the exiting vehicles leave the freeway, through vehicles move back into the right lane with a high lane-change frequency.

Not much research was found that dealt with capacity reduction in bad weather conditions, various lane widths and lateral clearances, and incidents. For instance, Highway Capacity Manual [17] mentions that a previous study found rain reduced freeway capacity by 14 percent, and 8 percent in another study.

According to the results concluded from previous studies [17], when the lane width was less than 12 feet, drivers were forced to travel laterally closer to vehicles on one adjacent lane. Drivers tended to compensate for this by maintaining longer spacing between their vehicle their leader's vehicle. The effect of restricted lateral clearance was similar to that of the narrow lanes. When roadside or median objects were too close to the pavement edge (less than 6 ft.), drivers would shy away from the pavement edge [17]. When drivers kept longer spacing between each other at a given speed, the accommodated volume or flow decreased.

One of the important adjustment factors in the HCM [17] is lane width and lateral clearance adjustment factor (f_w), and it had been in practice for decades. Nowadays f_w values are usually 1.00 in practice because new highway construction normally has adequate lane widths and lateral clearances, and this could also be the reason why very few recent reviews are available for f_w studies recently. FRESIM could not simulate freeways with restricted lane width or lateral clearance because there are no relevant adjustment factors considered in FRESIM.

A study on " Calibration of FREESIM to Achieve Desired Capacities" by Payne et. al. [31] found that the rubbernecking factor affected freeway capacity at incident locations and work zones. Capacity of a work zone or an incident area depends on (1) the number

of freeway lanes, (2) the number of blocked lanes (3) the rubbernecking factor (RF) parameter, (4) the duration of the incident and (5) the size of impact area.

Cragg and Demetsky [7] mention that the capacity reduction on a freeway found from previous studies, when one lane of a three lane facility was blocked by an incident, was appropriately 50%, rather than 33%. However, one can expect a minimum of 33% loss in physical capacity.

In freeway capacity analysis presented in the 1994 HCM [17], lane width and lateral clearance adjustment factor (f_w) and heavy vehicle adjustment factor (f_{HV}) tables were updated, and free flow speeds were used instead of design speeds.

In the HCM, f_{HV} is also a well developed adjustment factor with refined calibration methods and detailed estimation. The most comprehensive f_{HV} study was documented in a Federal Highway Administration Report [9]. Passenger Car Equivalents (PCEs) were used to convert a traffic stream composed of a mixture of vehicles into an equivalent traffic composed exclusively of passenger cars. The f_{HV} is computed from Equation (2-1), which is formulated in the HCM [17]:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \qquad (2\text{-}1)$$

where

E_T, E_R= passenger car equivalent for trucks/buses and recreational vehicles. and

P_T , P_R= proportion of trucks/buses and recreational vehicles respectively.

2.5 Other Related Studies

Traffic flow models are fundamental to capacity analyses because these models establish a theoretical base for the understanding of traffic stream characteristics. The basic findings were documented by May [24].

Schoen et. al. completed a project, NCHRP 3-45 titled Speed Flow Relationships on Basic Freeway Segments [28]. They were unable to address the factor of commuter vs. non-commuter traffic but they provided some interesting comparative data on the relationships between flow rates and average vehicle speeds in four different cities: San Diego, Sacramento, Seattle, and Des Moines. Although these data are unrelated to the driver population factor, it was interesting to note that the speed flow relationships among the four cities were significantly different.

Speed-volume curves are used to evaluate external factors that affect roadway capacities by many researchers. A recent study was performed by Brilon and Ponzlet [3] to evaluate the impact of weather conditions and a mixture of vehicles on roadway capacities. They found that the average traffic speed was lower in wet pavement conditions than in dry pavement conditions. The study also concluded that driving speeds were lower in leisure traffic, such as Sunday driving and during summer vacation season.

CHAPTER 3

IDENTIFY DEFICIENCIES OF FRESIM IN CAPACITY ANALYSIS

Although FRESIM is likely to be one of the best freeway simulation programs available, it does have some deficiencies and limitations in using it to approximate the maximum flow passing the analyzed freeway section, called freeway capacity. To help illustrate the problems identified in this chapter, four sections are organized to discuss all potential deficiencies . The four sections include:

(1) Factors Related to Driver Population,
(2) Factors Related to Vehicle Population,
(3) Factors Related to Freeway Geometry and weather conditions, and
(4) Other Pertinent Factors.

Many deficiencies identified in this chapter can be overcome by conducting additional calibration/validation and sensitivity analyses to further fine-tune the models. However, some of them need to be improved through major enhancements in FRESIM.

3.1 Factors Related to Driver Population

Microscopic simulation models, such as FRESIM, modeled traffic in an explicit manner in which each vehicle was treated as an entity based on car-following and lane changing logic. Individual driver attributes, such as the desired speeds, tolerate speeds, and the gaps to its leaders are dominated by the driver code. All identified deficiencies related to the driver attributes are listed and discussed below:

- Link Free Flow Speed
- Percentages of Cooperative and Non-Cooperative Drivers
- Driver Distribution
- Vehicle Tolerate Speed
- Car Following Sensitivity Factor
- Driver Reaction Time
- Hiatus Time Period
- Warning Sign
- Driver Lane Performance Factor
- Threshold Value of Urgency Factor

3.1.1 Link Free Flow Speed

1. Current Logic:

In the absence of any impedance, due to other vehicles or control devices, the link free flow speed means the free flow speed that is attained by traffic [6], (see the "Level of Service A" in the Highway Capacity Manual [17]). This information helps to determine vehicle desired speeds. The vehicle desired speed is determined by multiplying the link free flow speed and the factor of vehicle desired speed. Namely, the vehicle desired speed factor is the ratio of the vehicle desired speed to the link free flow speed. Vehicle desired

speeds are updated when vehicles enter new links, which is computed from Equation (3.1), formulated as follows:

$v_i = V_l Z_c S$, where (3.1)

i : vehicle identification number,

l : link identification number,

c: vehicle driver type

v_i : the vehicle desired speed of vehicle i on link l,

V_l: the link free flow speed of link l,

Z_c : the vehicle desired speed factor, and

S: the lane mean speed factor.

In FRESIM, the value of Z_c is fixed as (0.88, 0.91, 0.94, 0.97, 0.99, 1.01, 1.03, 1.06, 1.09, 1.12) for each driver code (1, 2, 3, 4, 5, 6, 7, 8, 9, 10). The value of the mean lane speed factor S varies with the lane where vehicles travel on. Therefore, for a vehicle on mainlines, right or left auxiliary lanes has different lane mean speed factors. However the lane mean speed factors are identical and equal to 1, the function of S is disabled.

2. Disadvantages:

Disadvantages of the current logic are detected after comprehensively investigating source code and simulation results. The disadvantages related to the link free flow speed are listed as follows:

(1) Fixed link free flow speed

The users specified link free flow speed for each link is fixed at all time periods until the simulation is terminated. This deficiency limits FRESIM by not being able to simulate several situations in freeway networks, such as:

(a) Traffic operations within work zones

Reduced speed limits in work zones are usually set for reducing speed variance, improving safety, and minimizing the accident rate while work zone activities are being performed. The changeable work zone speed limit could not be modeled.

(b) Variable speed limit signs

In real freeway networks, such as the NJ Turnpike, changeable speed limit signs are equipped for suggesting a proper speed limit that vehicles should follow. The speed limit is adjusted based on the level of congestion at the downstream links of the freeway segment. Because of the fixed link free flow speed, FRESIM cannot be used to analyze the impact variable speed signs have on traffic.

(2) Fixed vehicle desired speed factor Z_c .

As mentioned before, the value of Z_c varies between 0.88 and 1.12 with increments of either 0.02 or 0.03 for driver types from 1 to 10. Since the link free flow speed is highly dependent on the posted speed limit and the local driving behaviors, Z_c should be a function of the posted speed limit and the average link free flow speed. According to the current design, Z_c is suitable for most freeways with 55 mph as the posted speed limit and 65 mph link free flow speed. Therefore, the vehicle desired free flow speed ranges between 57.2 (e.g. 65 multiplied by 0.88) and 72.8 (e.g., 65 multiplied by 1.12) mph. In other cases, such as a freeway with 65 mph speed limit and 70 mph average link speed, the biased results may be generated because the vehicle desired speeds are still determined by the same Z_c. Table 3-1 shows the various vehicle desired speeds, determined by the link speed, and the driver code.

Table 3-1. Vehicle Desired Speed vs. Driver Type at Various Link Free Flow Speeds

			Link Desired Free Flow Speed			
Driver Code	%	Z_C *	55 mph	60 mph	65 mph	70 mph
1	10	0.88	48.40	52.80	57.20	61.60
2	10	0.91	50.05	54.60	59.15	63.70
3	10	0.94	51.70	56.40	61.10	65.80
4	10	0.97	53.35	58.20	63.05	67.90
5	10	0.99	54.45	59.40	64.35	69.30
6	10	1.01	55.55	60.60	65.65	70.70
7	10	1.03	56.65	61.80	66.95	72.10
8	10	1.06	58.30	63.60	68.90	74.20
9	10	1.09	59.95	65.40	70.85	76.30
10	10	1.12	61.60	67.20	72.80	78.40

* Z_C : vehicle desired speed factor.

After simulating the 55/65 (posted speed limit/link free-flow speed) case, the observed average link speed may be similar to the speed of the real world case. However, detailed information, such as the vehicle speed variation, could not be observed from the simulation output; which might show a great difference once compared against the data collected from the real world networks.

(3) Disabled lane mean speed factor

According to previous field studies [37], the average vehicle speed in the left most lane was usually faster than speeds in the remaining right lanes, because vehicles usually use the left lane for bypassing their leaders traveling at lower speeds. This situation can be replicated quite well by the car following model in FRESIM.

The lane mean speed factor S is equal to 1 at any situation. Therefore, the vehicle desired speed is determined by the link free flow speed and the questionable Z_c. The lane mean speed factor will not affect vehicle desired speeds. One suspected

deficiency existing in FRESIM is that the average lane speed in all lanes are very close. In some situations, simulation results may be even more unreasonable. For example the average lane speed of the left most lane may be lower than that in the right lanes, because of slow vehicles traveling in the left most lane. However, the majority of slow vehicles rarely use the left most lane in multi-lane freeway networks. This issue needs to be further tested.

3.1.2 Percentages of Cooperative and Non-Cooperative Drivers

1. Current Logic:

The percentage of cooperative drivers who will yield their right-of-way for other vehicles is specified in card type seventy. Whenever a vehicle is trying to change lanes, the cooperative driver code of its putative follower is checked. If the putative follower is a cooperative driver, the vehicle has a greater chance to shift to its desired lane, because the risk value is smaller as the vehicle meets a cooperative follower. The procedures to determine the risk value for a lane changer are listed as follows:

Step 1	Identifying the vehicle that has desire to change lane
Step 2	Checking the putative leader of the vehicle identified in Step 1. If there is no putative leader, go to Step 3; otherwise, go to Step 4
Step 3	If there is no putative leader, generate a random number to determine whether the vehicle will meet a cooperate putative follower who will yield its right of way for the vehicle.
Step 4	If there is a putative leader and if the vehicle meets a cooperative leader and follower, assign the lowest risk value (R_{co} = -10) to the vehicle; otherwise assign the higher risk value (R_{co} = -8) to the vehicle. The lower risk value provides better probability for making a successful lane change because a safer gap will be given if the current acceleration rate of the vehicle is greater than the risk value.

2. Disadvantages:

(1) The percentage of cooperative drivers who will yield their right-of-way is generally implemented in every link if users specify the percentage of cooperative drivers in card type seventy. For example, if the user specifies 60% of cooperative drivers, this percentage is identically implemented in all links; although this could not reflect the heterogeneous distribution of the driver population. The user specified percentage of cooperative drivers is highly related to the total number of vehicles that can successfully change lanes on a link, especially in congested traffic conditions. Theoretically, if the percentage of cooperative drivers increases, the total number of lane changes should increase. This assumption needs to be further tested and verified by a series of simulation runs.

(2) The risk value of a potential lane changer is determined in an arbitrary manner. The risk value is constrained by two fixed values of Z_{MX} and Z_{MN}, where Z_{MX} and Z_{MN} are the maximum deceleration rates for emergency and non-emergency cases, respectively. Due to the fixed Z_{MX} and Z_{MN} for all driver types, the impact of these values in the number of lane changes is suggested to be analyzed. We suspect that the values of Z_{MX} and Z_{MN} will increase or decrease the total number of lane changing to a certain extent, depending on the driver population distribution and the level of congestion on the analyzed roadways.

(3) A suspected program bug is detected (in Subroutine FGAP) during this investigation. According to the current logic if a vehicle meets a cooperative driver, the risk value R_{CO} is -8; otherwise, the risk value is -10. This assignment is contradictory to the definition of the risk value. In fact, R_{CO} should be assigned to a vehicle meeting its follower who is a cooperative driver. Since this vehicle gets less risk value, the probability of a successful lane change increases.

3.1.3 Driver Distribution

1. Current Logic:

The FRESIM model allows ten different driver types defined from the cautious to the aggressive [6]. Drivers of each driver type occupy 10% of the total driver population. The driver type of a vehicle is determined by a random process when the vehicle is ready to be emitted onto a network. According to this design, one phenomenon should be observed. The driver population is uniformly distributed over ten driver types.

2. Disadvantages:

(1) According to the current design, the driver population is generated based on a uniform distribution. The first question is raised from a realistic point of view for the fixed driver distribution in FRESIM. Because of this assumption, FRESIM produces one of many driver combinations which represents limited patterns of driving behaviors. This limitation restricts FRESIM's capability to properly model existing freeway networks.

(a) Driver distribution might be highly related to the types of vehicles. For example, truck drivers experience pressure from their scheduled arrival time for delivering goods to their destination. This factor may increase the aggressiveness of a truck driver while on the road. Therefore, it is beneficial to allow the user to adjust the driver type distribution in truck fleets.

(b) If the vehicle desired speed variation increases the freeway capacity decreases, especially on one lane freeways, because passing is not possible. The adverse effect of trucks, and other slow moving vehicles, reduces the freeway capacity and increases the total delay. The driver population may follow other distributions

over the ten driver types (i.e., normal and gamma distributions). Distribution of the driver population does affect the traffic operations and capacity on freeways [4]. A preliminary test was performed in the study, and differences in capacity and average speed of freeway links were presented.

(2) The driver distribution should be location dependent. In residential areas, driving behaviors may be more conservative than near a Central Business District (CBD). We suggest that FRESIM developers consider the feasibility for users to calibrate driver behaviors corresponding to local land use characteristics. Since FRESIM generates an identical driver population for different freeway networks, results generated from the FRESIM model may be questionable.

(3) The driver distribution will affect several important modules for handling vehicle operations in FRESIM. These modules are:

(a) to determine the urgency factor when vehicle try to make lane changing
(b) to distribute the car-following sensitivity factors over the driver population
(c) to calculate the vehicle tolerate speeds
(d) to calculate the vehicle desired free flow speeds
(e) to compute the acceptable risk value for a vehicle trying to change lane, and
(f) to check the driver's desire to change lane

3.1.4 Vehicle Tolerate Speed

1. Current Logic:

A vehicle's tolerate speed is immediately determined when the vehicle desired free flow speed has been calculated. The role of vehicle tolerate speed in vehicle attributes is a reference speed for computing its desire to change lanes. The vehicle tolerate speed is determined based on Equation (3.2). From Equation (3.2), the vehicle tolerate speed

deviates from the vehicle desired speed according to the driver type and two parameters, called x and y. The parameters x and y are fixed (0.5 and 0.02, respectively) and not modifiable. The tolerate speed of vehicle i is formulated as follows:

$$v_{t,i} = v_i(x + c_i\, y) \qquad (3.2)$$

i: the vehicle identification number

$v_{t,i}$: the vehicle i's tolerate speed,

v_i: the vehicle's desired free flow speed,

c_i: the driver type of vehicle i, and

$x = 0.5$, and

$y = 0.02$

2. Disadvantages:

Regardless of deficiencies in driver distribution described in section 3.1.3, the procedures to determine the vehicle tolerate speed are also questionable. From Equation (3.2), the tolerate speed ranges between 0.52 and 0.7 times of the vehicle desired speed, depending on the driver code. For example, if v_i is 57.2 mph, the driver code of vehicle i is 1, shown in Table 1. The tolerate speed of this driver is 57.2 (0.5 + (1) (0.02)) = 29.7 mph. However, if v_i is 72.8 mph, the driver code of the vehicle is 10. The tolerate speed is 72.8(0.5 + (10)(0.02)) = 50.9 mph. Therefore, the vehicle tolerate speed varies in a wider range than the vehicle desired speed. This is because the vehicle tolerate speed is a function of the square value of the driver code. Currently, the fixed values of x and y are 0.5 and 0.02 respectively, while c_i varies from 1 to 10. However, are these parameters suitable for all freeway networks? The answer is unknown and probably negative. More tests should be performed in the near future. This is an important deficiency that needs to be resolved in analyzing freeway capacity, because the parameters directly affect the vehicle tolerate speeds. Moreover, the vehicle tolerate speed affects the average link speed, the serviced flow rate, and the desire of a driver to change lanes. Hence, the parameters of x and y are suggested to be carefully calibrated by users.

3.1.5 Car-Following Sensitivity Factor

1. Current Logic:

The car following sensitivity factor is defined by a time gap representing the minimum gap retaining between a vehicle and its leader. The default values of car-following sensitivity factors range from 1.5 to 0.6 seconds for the driver code from 1 to 10, respectively. The value of the car following sensitivity factors can be calibrated by users through card type sixty-eight. The vehicular operations will not violate the factor during the simulation. Generally, a vehicle will accelerate until its desired free flow speed or the minimum time gap to its leader is achieved.

2. Disadvantages:

(1) In the real world, driver behaviors vary with various factors, such as the minimum acceptable gap to its leader in different situations. For example, the time gap, called the car-following sensitivity factor between vehicles is not a constant over time and space. The car following sensitivity factor designed in FRESIM varies with the driver code but other elements affecting the values of the factors are neglected. The major concerns include (especially in traffic-jam conditions) the values of the sensitivity factor that should be different (i.e., smaller) from that of normal traffic conditions because most drivers may have been more cautious in congested situations.

(2) Secondly, the sensitivity factors will be adjusted by the rubbernecking factor when vehicles travel in work zones or incident areas. However, the value of the sensitivity factor will not be adjusted if vehicles travel on the freeway sections with grade terrain. According to these findings, we believe that the current design of the car following sensitivity factors does have room to be enhanced. In real world traffic

operations, such as the geometry at grade terrain and at congested conditions, can be modeled better now.

3.1.6 Driver Reaction Time

1. Current Logic:

The driver response lag time is classified into two categories, including the response lag times for acceleration (t_{AC}) and deceleration (t_{DC}), which are lags or time delays that motorists experience when making required maneuvers. The default response lag times for acceleration and deceleration are both equal to 0.3 seconds and modifiable by users in card type 69. The procedures to determine the proper t_{AC} or t_{DC} for each driver in FRESIM are listed as follows:

Step 1 Comparing the current vehicle acceleration a_c and that in the previous time step a_p. If $a_c > a_p$, go to step 2; otherwise, go to step 3 if $a_c < a_p$, or step 4 if $a_c = a_p$.

Step 2 The response lag time is t_{AC}.

Step 3 The response lag time is t_{DC}.

Step 4 The response lag time is zero.

The response lag time affects the safe distance being kept between a pair of vehicles (i.e., a vehicle and its leader). If the response lag time is long, the safe distance kept for avoiding collision between vehicles will be long, and the density on the freeway segment will reduce. Therefore, the freeway capacity decreases. On the other hand, if the response lag time is short, the freeway capacity should increase.

2. Disadvantages:

(1) In real traffic situations, the response lag time varies with differing situations (e.g., driver types, locations, and time periods). In the current FRESIM study response lag

times for both acceleration and deceleration are constant, and implemented identically, during any location and time period. This deficiency causes an impact in several modules when FRESIM processes vehicle movements, such as:

(a) Adjusting vehicle acceleration and deceleration rates
(b) Avoiding collision for a vehicle and its leader or follower
(c) Determining travel distance in a time step
(d) Moving the follower vehicle
(e) Computing the gap between the vehicle and its follower
(f) Processing vehicles approaching an incident blockage
(g) Computing vehicles' new speed and displacement

(2) The time lag for a vehicle which was accelerating at the previous time step should be very small (at least less than 0.3 seconds) if the vehicle is accelerating continuously at the current time step. The same idea should sustain in deceleration cases. The current design may underestimate the time for a vehicle accelerating or decelerating to a particular speed.

3.1.7 Hiatus Time Period

1. Current Logic:

The current hiatus period is fixed (e.g., 3 seconds) and not modifiable. The determination for the value of the hiatus period (t_h) is based on an assumption that a lane changing maneuver will take the average t_h seconds. The lane changing status of vehicle i is represented by t_{ci}. If t_{ci} is greater than zero, it means that the vehicle i does not yet complete its lane changing maneuver. During the hiatus time period, the vehicle will not consider another lane changing action. Moreover, the follower of a vehicle will not consider a lane change maneuver if the vehicle is within the hiatus period. That means

FRESIM will not allow a pair of vehicles (i.e., a vehicle and its follower) to change lanes simultaneously.

2. Disadvantages:

(1) The duration of the hiatus period is one of the most important factors which greatly affects the number of lane changes on freeway links. A major limitation in FRESIM is that a pair of vehicles (i.e., a vehicle and its follower) could not simultaneously execute lane changing maneuvers. However, this type of lane changing operation does exist in most freeway networks.

(2) Duration of the hiatus period should differ according to locations and should be calibrated by users. Theoretically, the duration of the hiatus period varies with several factors, such as vehicle types and driver types of the lane changer and its leader, and the level of traffic congestion. For example, trucks and buses will take a longer time changing lanes than passenger cars. Aggressive drivers need less time for changing lanes than timid drivers.

After investigating the FRESIM source code and simulating a data set with different hiatus durations, we found that if the duration of the hiatus period increases, the average number of lane changing decreases. As mentioned before, the freeway capacity is influenced by the number of lane changes. Therefore, the impact from the hiatus duration to traffic operations in the simulation model is substantial. Additionally, the observed maximum flow rate traversing on a freeway section might be underestimated if the hiatus period duration is higher than the realistic hiatus time duration. Several modules in FRESIM that were affected by the duration of the hiatus period are listed below:

(a) Adjusting vehicle acceleration and deceleration rates
(b) Determining gap between a vehicle and its follower

(c) Checking lane changing status of a vehicle

(d) Moving the lane changer to its target lane

(e) Determining the entering lane and the leading vehicle of a emitted vehicle, and

(f) Moving the follower and leader of a vehicle

3.1.8 Warning Signs

1. Current Logic:

The location of a warning sign is the position of a vehicle at which the vehicle begins to respond to a condition, usually represented by an object (e.g., an off-ramps). The FRESIM model allows only one warning sign and serves one object. Few objects in FRESIM need to have their own warning signs to alert vehicles moving into their proper lanes. Some warning sign locations are modifiable, such as the warning signs for off-ramps, incident blockages, and lane drops. Since the number of warning signs associated with one object (i.e., off ramp) are usually greater than ones in most existing freeway networks, the warning sign locations specified in FRESIM should not be the actual warning sign location in the real world. It is very difficult to model actual lane changing behaviors if only one warning sign is allowed for off-ramps. Some warning sign locations are not modifiable, such as warning signs for alerting vehicles reacting to congestion in downstream on-ramps.

The purpose of warning signs in both the freeway simulation model as well as those in existing freeway networks is to inform vehicles that they should move to their designated lanes. Therefore, the location of warning signs is the place where vehicles that are not in their proper lane to change lanes if there is a safe gap.

2. Disadvantages:

The reactions of drivers to a warning sign are very difficult to be modeled. A driver's reactions may vary with the distance between the object (say an off-ramp) and its warning sign, the driver type (commuter or non-commuter), and the level of congestion. Not every driver passing a warning sign will move to her/his designated lane, even if there is a safe gap in that lane. This situation can be observed from existing freeway networks in both congested and non-congested traffic conditions. Another problem is related to the lane changer distribution before an off-ramp. In reality, a specific off ramp may be served with multiple warning signs. Thus, real world lane changing activities may distribute very differently than that modeled in FRESIM, because FRESIM uses one warning sign for guiding vehicles to their destined lanes. This limitation causes the majority of vehicles destined to an off ramp to immediately shift to their designated lane after they pass a warning sign in permitting traffic conditions. From Figure 3-1a, a bottleneck reaction might be detected immediately after the warning sign if there are many vehicles shifting from left to the right lanes. Comparing with the real world situation, shown in Figure 3-1b, the flow passing the real freeway segment may be smoother than the simulated one because the number of mandatory lane changes are not concentrated after the first warning sign. Thus, we suspect that FRESIM might underestimate the capacity of a freeway section that contains off ramps served with multiple warning signs.

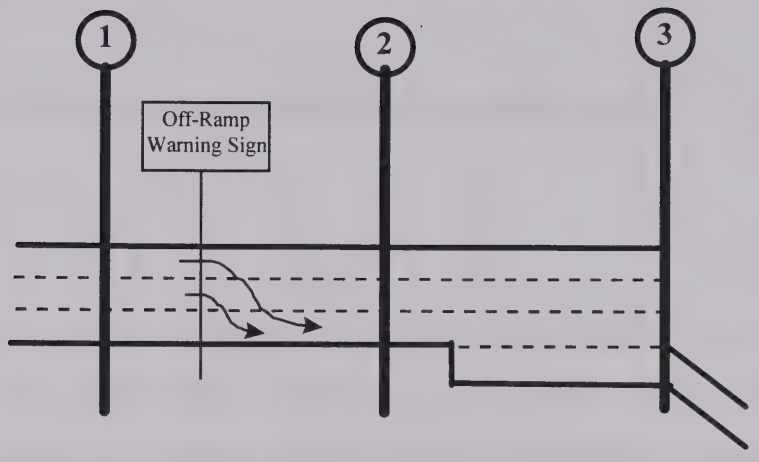


Figure 3-1a Mandatory Lane Changes in FRESIM

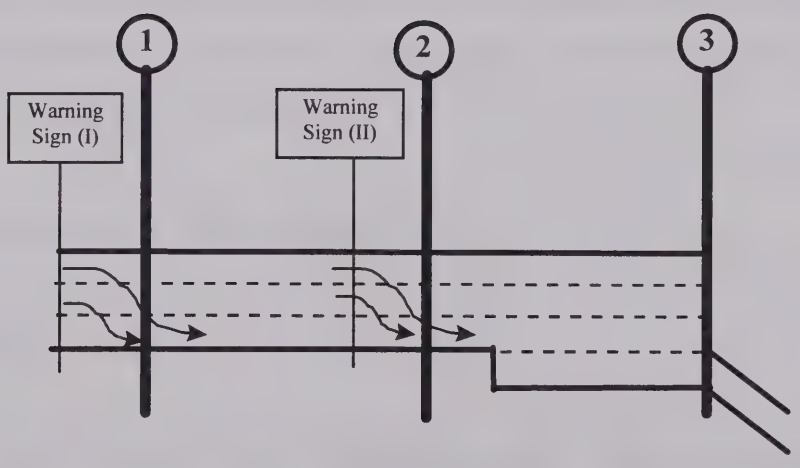


Figure 3-1b Mandatory Lane Changes in a Real Freeway Segment

3.1.9 Driver Lane Preference Factor

1. Current Logic:

This feature is not available in the current FRESIM simulation model.

2. Disadvantages:

The lane performance factor can be defined as the desire for a vehicle to stay in a specific lane. According to the current design of FRESIM, the lane preference for all driver and vehicle types are identical. For example, on a three lane directional freeway link, vehicles travel on all lanes with equal preference. The issue raised here exposes the needs to introduce the innovative lane preference factor for every driver type and every vehicle type for handling vehicle operations properly. Most non-aggressive drivers will not use the left most lane except for bypassing slow vehicles and blockages, or in traffic-jam conditions. Therefore, at certain levels of congested conditions, the average travel speed of the left lane should be higher than the speed of the right lanes. Other situations, such as bad pavement conditions (i.e., pot holes or snow), can be better modeled now if the lane performance factor is built in FRESIM.

3.1.10 Threshold Value of Urgency Factor

1. Current Logic:

The threshold value of urgency factor is embedded in the FRESIM model for every driver type. The default values are (0.95, 0.9, 0.85, 0.8, 0.75, 0.7, 0.65, 0.6, 0.55, 0.5) for driver types 1 through 10, respectively. The more aggressive the driver type, the higher the threshold urgency factor. The role on the urgency factor in FRESIM is providing the proper risk value when vehicles desire to change lanes. If the urgency factor for a discretionary lane changer is greater than 1, the risk value of the vehicle is set to be Z_{MX}.

On the other hand, if the urgency factor is less than 1 and if the vehicle's urgency factor is less than its threshold value, the risk value is set to be Z_{MN}. However, if the urgency factor is less than 1 and the vehicle's urgency factor is greater than the threshold urgency factor, the risk value is determined based on an assumption that the risk value decreases linearly from Z_{MX} to Z_{MN} if the vehicle urgency factor increases. The risk value can be obtained from the equations formulated as follows:

$$R = Z_{MX} \quad \text{when } U_v \geq 1 \tag{3.3}$$

$$R = Z_{MN} + (Z_{MX} - Z_{MN})\left(\frac{U_v - U_t}{1 - U_t}\right) \quad \text{when } U_t < U_v < 1 \tag{3.4}$$

$$R = Z_{MN} \quad \text{when } U_v \leq U_t \tag{3.5}$$

R : the risk value

Z_{MN} : the maximum deceleration rate at non-emergency condition (-8)

Z_{MX} : the maximum deceleration rate at emergency condition (-15)

U_v : the vehicle urgency factor

U_t : the threshold vehicle urgency factor

The risk value for each potential lane changer is updated and checked at every time step. If the vehicle acceleration rate is greater than the risk value of the vehicle, a safe gap exists. Otherwise, the gap that the vehicle watched is an unsafe gap.

2. Disadvantages:

The risk value of a potential lane changer is determined by the maximum vehicle deceleration rates at emergency and non-emergency conditions, the vehicle urgency value, and the threshold vehicle urgency factor. Since the value of the threshold urgency factor is not allowed to be calibrated, the users do not have the capability to modify the risk values if the simulated results could not reflect the realistic traffic operations (e.g., the abnormal number of lane changing in the analyzed freeway section).

3.2 Factors Related to Vehicle Population

The vehicle characteristics affecting the freeway capacity are discussed in this section. Six deficiencies are identified and discussed, which are listed below:

- Percentages of Fleet Components
- Truck Turn Movements (Specific Truck O-D Matrix)
- Minimum Separation for Generation of Vehicles
- Vehicle Maximum Deceleration Rate
- Mean Start-up Delay
- Truck Movements before Work Zone or Incident Area

3.2.1 Percentages of Fleet Components

1. Current Logic:

The percentage for every vehicle type in each fleet component is fixed and not modifiable in FRESIM. A vehicle code is assigned based on a random sampling of the default fleet component distribution shown in Table 3-2. The code of a vehicle affects several operational vehicular characteristics; such as the vehicle length, maximum acceleration and deceleration rates, and jerking. These characteristics can be calibrated by users.

Table 3-2 Default Values for Each Vehicle Type

Fleet	Vehicle Type	Length	Percentage
Passenger Cars	1 low performance passenger Car	17	**25**
	2 high performance passenger car	19	**75**
Trucks	3 single-unit truck	38	**31**
	4 semi-trailer truck with medium load	56	**24**
	5 semi-trailer truck with full load	56	**36**
	6 double bottom trailer truck	67	**9**
Transit Vehicles	7 inter-city buses	43	**100**

The timing for emitting vehicles from an entry link is determined based on the flow rate of the entry link and the percentage of trucks. However, the further detailed vehicle type distribution of each fleet is determined internally. The logic for determining vehicle type of an emitted vehicle is stated as follows:

Step 1 checking the timing if a vehicle is ready to be emitted from an entry link.

Step 2 determining the vehicle fleet based on the percentages of fleets.

Step 3 determining the type of the vehicle based on information shown in Table 3-1. For example, if the vehicle is a passenger car, it is a high performance car with 75% probability or it is a low performance car with 25% probability.

2. Disadvantages:

(1) The percentages of vehicle types in each fleet should not be fixed. Since vehicles are distributed differently in vehicle types over space and time, this information embedded in FRESIM can not reflect the realistic vehicular population in most freeway networks. The link specific and time varied vehicle type distribution is suggested to be enhanced.

(2) The default operational vehicle characteristics, such as the maximum deceleration rates at both emergency and non-emergency conditions (e.g., . -15 ft/sec^2,and -8 ft/ sec^2), and jerking (e.g., 7 ft/sec^2) values are identical for all vehicle types. Although these values can be modified by users, the more realistic default values are suggested to be replaced based on the operational performance of all vehicular types.

(3) The current default transit vehicles are represented by 43 feet inter-city buses. Since the size of transit vehicles are variant (e.g., small buses, normal buses, and articulated buses), the number of types of transit vehicles should be increased. This will enable the simulation model to properly reflect the actual transit vehicle operations on urban freeway networks.

3.2.2 Truck Turn Movements (Specific Truck O-D Matrix)

1. Current Logic:

FRESIM allows users to specify vehicle turning percentages for every link that has an off-ramp emanating from its downstream end. If a link has only a through movement, this record can be omitted, and the model will assign 100% turn percentage to the through link. Since the specification of vehicle turn movements in card type 25 could not provide enough information to generate the anticipated vehicle operations, especially within the weaving sections, card type 74 has been designed for specifying vehicle origin-destination information to overcome this deficiency. The information in card type 74 simply overwrites the information specified in card 25. Thus, the vehicle destination is determined based on the origin-destination information (if any), rather than the turn movement information. However, the information is used for assigning destinations to the general vehicle population only for a specific vehicle fleet. Users would not be able to simulate such situations; such as a certain amount of vehicles (for a specific vehicle type)

exiting in off-ramps. For example, the user could not simulate 75% of trucks entering from node 1 and exiting from node 3 as shown in Figure 3-2.

2. Disadvantages:

The design of card types 25 and 74 continue to have limitations in handling vehicle percentage information by vehicle types (such as an unreasonable number of trucks in weaving streams within a freeway weaving section). Because of significant traffic impacts (i.e., reduced capacity, increased average delay, and decreased average travel speed) incurred by the number of trucks traveling on freeway networks, the turn movement and origin-destination demand information for truck fleets are suggested to be enhanced for freeway capacity analysis. Generally, trucks transport goods from an origin point to their destination, and trucks generate greater impact on highways then passenger cars. It is improper to treat trucks as passenger cars when assigning destinations to vehicles. In handling vehicle turn movements on freeways, the O/D demand of trucks could not be modeled because FRESIM was not capable of controlling a percentage of heavy vehicles exiting on off-ramps. No matter what the vehicle population distribution, the destination of trucks and passenger cars are assigned based on one O/D trip table generated by the FRESIM preprocessor.

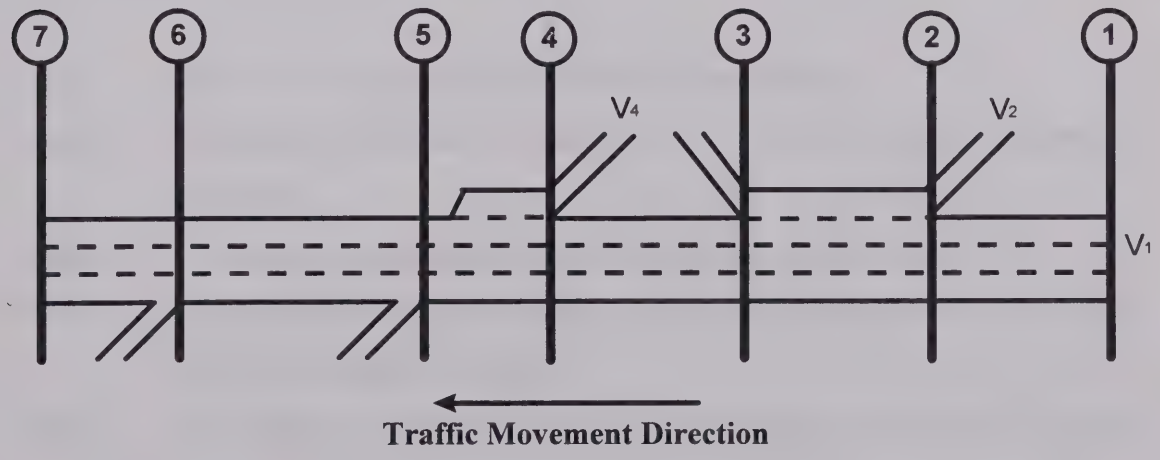


Figure 3-2 Geometric Diagram of a Freeway Segment

In Figure 3-2, for example, V_1 is the entry volume from node 1, while V_2 and V_4 represent the entry flow from on-ramps at node 2 and 4. In card type 25, if the vehicle turn movement percentage on link (2,3) is 20% exiting at node 3 with 80% of the total flow, we could not guarantee an expected amount of trucks exiting to node three. Similarly, the origin-destination information in card type 74 could not guarantee the same situation. Link (2,3) of the freeway segment, shown in Figure 3-2, is a typical weaving section. If we could not simulate a correct number of trucks exiting through the off-ramp emanating from node three, the observed maximum flow (capacity) in that section might be inaccurate.

3.2.3 Minimum Separation for Generation of Vehicles

1. Current Logic:

The default value of the minimum separation for generation of vehicles is 2 seconds, named H_{MN} . This parameter governs the maximum rate at which a vehicle can be emitted onto the analyzed network. Based on the default value, the maximum entry flow is 1800 vehicles per hour per lane (vphpl). The current vehicle emitting procedures are investigated and listed in the following steps:

Step 1	Checking whether a vehicle is ready to be emitted.
Step 2	Checking which lane is available for the vehicle and find the leader on the lane.
Step 3	Computing the headway between the vehicle and its leader.
Step 4	If the headway is less than the H_{MN}, this lane is not qualified, then go to step 2; otherwise, go to step 5.
Step 5	Emitting this vehicle onto the designated lane. If no lane is qualified, put this vehicle in a queue behind the entry link.

2. Disadvantages:

The disadvantage of the vehicle emission logic is that the default value of the minimum separation for the generation of vehicles is improper in most congested freeway networks. Lots of previous studies [17, 25] presented that the flow on the freeway segment could be much greater than 1800 vphpl (2200 pcphpl on 4-lane freeways, and 2300 pcphpl on 6-lane or 8-lane freeways). This logic force the vehicles emitted from all entry links to follow the same restriction rule; which is the headway between the emitted vehicle and its leader has to be less than H_{MN}. This logic usually causes much vehicle backup behind some entry nodes with large entry volumes. The biased simulation results might be generated because of the total anticipated flow failing to be emitted onto the network. Although this value can be calibrated by users, the problem still exists. For example, if users specify that the minimum separation is 1 second, it is unquestionable that FRESIM can generate the maximum 3600 vphpl onto freeways for every entry link. However, this value is not suitable for the entry links with low entry volumes. The value of H_{MN} is suggested to be entry link specific and time dependent, in order to reflect the actual patterns of vehicles that have been emitted onto the networks. The vehicle emission logic should be further checked to minimize the potential deficiencies.

3.2.4 Vehicle Maximum Deceleration Rate

1. Current Logic:

The default values of the maximum deceleration rates under emergency (Z_{MX}) and non-emergency (Z_{MN}) environments, are given as -15 and -8 ft/sec^2, respectively. The maximum deceleration rates for all vehicle types can be modified in card type 71 but still the deceleration rates are fixed and identical for all vehicles regardless what the vehicle type is. The emergency deceleration rate is used when processing vehicles approaching an incident blockage, while the maximum non-emergency deceleration Z_{MN} is used for determining (1) the minimum acceptable mandatory risk when processing vehicles trying

to change lanes, and (2) the maximum deceleration rate when vehicles are approaching meters or joining a queue.

The value of vehicle jerking is the change of acceleration per unit of time. It is an important factor for passenger riding comfort, and also helps to determine how long a vehicle can accelerate to its desired speed. The maximum jerk value is used to predict whether a vehicle can achieve its desired speed at the end of the current time step.

2. Disadvantages:

(1) Currently, the maximum deceleration rates of all vehicle types are identical. This design is conflicting with the characteristics of vehicle performance. For example, the maximum vehicle deceleration rates are different between heavy vehicles and passenger cars.

(2) The jerking value is used for checking whether the adjusted vehicle acceleration rate exceeds the vehicle maximum acceleration rate. Currently, a vehicle's maximum acceleration rate is adjusted when the vehicle is traveling on a grade link but the jerk value of each vehicle remains the same. This deficiency may cause FRESIM to overestimate the time needed by a vehicle to achieve its desired speed. We suggest that the vehicle jerk value should be adjusted at grade geometry.

3.2.5 Mean Start-Up Delay

1. Current Logic:

The default value of the link specific mean start-up delay is 1 second, which is used when FRESIM processes vehicles to pass meters on ramps. Even the value of mean start-up delay can be adjusted by users, but it is identical for every vehicle type. This deficiency might generate effects when evaluating traffic flow passing through meters. Additional

impact related to the duration of start-up delay, and the metered flow, should be further analyzed.

2. Disadvantages:

Not every vehicle's start-up delay will be the same. For example, a non-aggressive driver using a heavy vehicle usually has a longer start-up delay because of the cautious driving behaviors, and the vehicle operational characteristics. Since this factor only affects flow on metered ramps, the mainline traffic operations will not be significantly influenced. We suggest that the duration of the delay should vary with at least driver types and perhaps vehicle types.

3.2.6 Truck Movements Before Work Zones or Incident Areas

1. Current Logic:

All vehicles passing an incident warning sign, will perform mandatory lane changes if the vehicles are in front of the blocked lane. Simulation results demonstrated that a truck could not shift to its designated lane. This situation happened if the truck was the first vehicle before the blocked lane, no matter how high the percentage of cooperative drivers were specified. The truck would stand still until the blocked lane was reopened or a safer gap emerged.

2. Disadvantage:

In the real world, vehicles on lanes usually take turns to use the opened lane in front of a work or an incident zone. It is very unusual that trucks stay long before a blocked lane without being able to shift to the opened lane.

3.3 Factors Related to Freeway Geometry and Weather Conditions

The factors related to freeway geometry and weather conditions, affecting freeway capacity, are identified and listed as follows:

- Weather conditions (i.e., fog, snow, rain, etc.)
- Lane width and lateral clearance
- Warning signs for grade geometry
- Risk factor

3.3.1 Weather Conditions

1. Current Logic

FRESIM can simulate various pavement conditions, such as concrete and asphalt in both dry and wet conditions. The pavement code determines the friction coefficient used in Equation (3.6) for determining the safe vehicle operating speed.

$$V = \sqrt{15R(e+f)} \tag{3.6}$$

where,

V: vehicle speed (mph)

R: radius of curvature

e: super-elevation rate

f: friction coefficient

Currently, the friction coefficients for all pavement conditions are identical and equal to 0.16, regardless if the pavement is dry or wet, concrete or asphalt. Therefore, if users do not calibrate the friction coefficient values at card type 20, the resulting vehicle speed will not be changed.

2. Disadvantage:

Currently FRESIM can not handle vehicle operations at bad sight distance conditions. Therefore, the impact caused by bad weather conditions (i.e., fog, snow, and rain) could not be observed from simulation results.

3.3.2 Lane Width and Lateral Clearance

1. Current Logic:

This feature is not available in FRESIM

2. Disadvantage:

According to the results concluded from the previous studies [17], when the lane width is less than 12 feet, drivers are forced to travel laterally closer to the one on the adjacent lane than they normally drive. Drivers tend to compensate for this by maintaining longer spacing with their leaders. The effect of restricted lateral clearance is similar to that of narrow lanes. When roadside or median objects are too close to the pavement edge (less than 6 ft.), drivers will shy away from them, positioning themselves from the pavement edge, rather than under normal or ideal conditions [17]. When drivers keep longer spacing between vehicles at a given speed, the accommodated volume or flow decreases. Thus lane width and lateral clearance do affect freeway capacity; however, this factor could not be modeled by FRESIM.

3.3.3 Warning Signs for Grade Geometry

1. Current Logic

This feature is not available in the current FRESIM

2. Disadvantages

For vehicles moving on a freeway section at grade terrain, the low speed vehicles (i.e., trucks, buses, and some low performance cars) tend to shift to the outer lanes. Some of those vehicles even shift to the outer lanes before the grade section if those vehicles are informed in advance by a warning sign. In most existing freeway networks there usually is a warning sign in front of grade terrain geometry to alert low performance vehicles to vacate the left lanes for faster vehicles. Currently, FRESIM can not simulate this situation because there are no warning signs designed for alerting those vehicles. The lack of warning signs causes heavy or slow vehicles to continue using left lanes in grade sections, which may result in greater overall delays. Under those situations, the approximate capacity might be underestimated.

3.3.4 Risk Factor

1. Current Logic:

If there is a vehicle passing an off-ramp warning sign and is trying to make a mandatory lane change, the risk value will be calculated for determining whether the gap on the candidate lane is safe or not. The procedures to determine the risk value for a lane changer is shown below:

Step 1 Identifying a vehicle passing an off-ramp warning sign and the off ramp is the vehicle's destination

Step 2 If the vehicle's current location is between the warning sign and 200 feet after the warning sign, the risk value is Z_{MX}, where Z_{MX} is the maximum vehicle deceleration rate at emergency condition.

Step 3 If the vehicle location is 200 feet after the warning sign, the risk factor increased non-linearly from Z_{MN} to Z_{MX}.

The concept of the increasing acceptable risk to the lane changer is formulated and suitable for various situations. The first situation is if the current speed of the lane changer is less than 5 ft/sec, Z_{MX} assigned to the vehicle as its risk value, shown in Equation. (3.7).

$$R = Z_{MX} \tag{3.7}$$

If the current speed of the lane changer is greater than 5 ft/sec, the risk value of the vehicle is determined based on the lane that the vehicle currently occupies. If the vehicle is already on the deceleration lane, its risk value can be obtained from Equation. (3.8):

$$R = Z_{MN} + (Z_{MX} - Z_{MN}) \cdot \sqrt{\frac{d_1}{L_1}} \tag{3.8}$$

Where d_1 is the distance between the vehicle and the beginning of the deceleration lane before the off-ramp, and L_1 is the length of the deceleration lane before the off-ramp shown in Figure 3-3. Figure 3-4 shows the relationship between the risk values and the vehicle location.

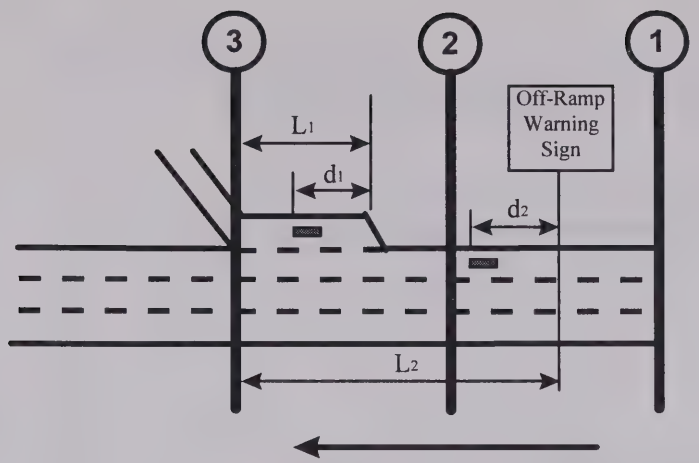


Figure 3-3 Geometric Diagram of An Off-Ramp and Its Warning Signs

2. Disadvantage:

If the deceleration lane length is 1000 ft, Figure 3-4 shows the risk value for a vehicle passing the location of an off-ramp warning sign at various conditions. In that figure, the R1 curve represents the risk value when the vehicle's speed is lower than 5 ft/ sec, while R2 or R3 show the risk value when the vehicle is on the auxiliary lane or mainline, respectively. It is questionable to assign Z_{MX} to a vehicle as its risk value when the vehicle just passed the off-ramp warning sign shown in curve R3. This problematic logic implies that many vehicles move to the right lane when these vehicles just passed the off-ramp warning sign, because the assigned risk values are very low. We are not sure whether this kind of model is better to reflect the realistic traffic operations in this condition. We believe this model should be calibrated by users themselves after comparing the results generated by the simulation model and the data collected from the analyzed site.

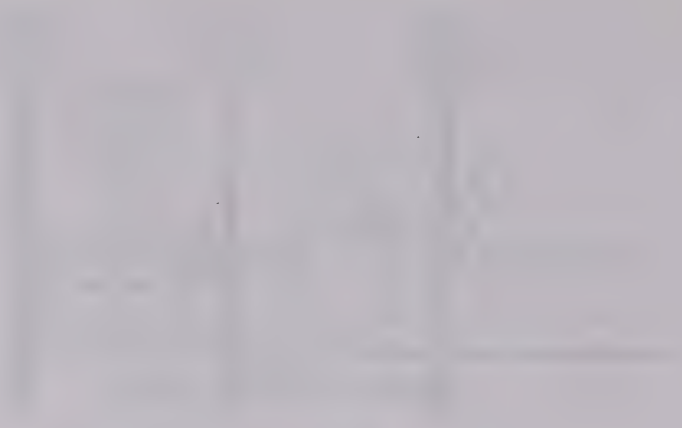
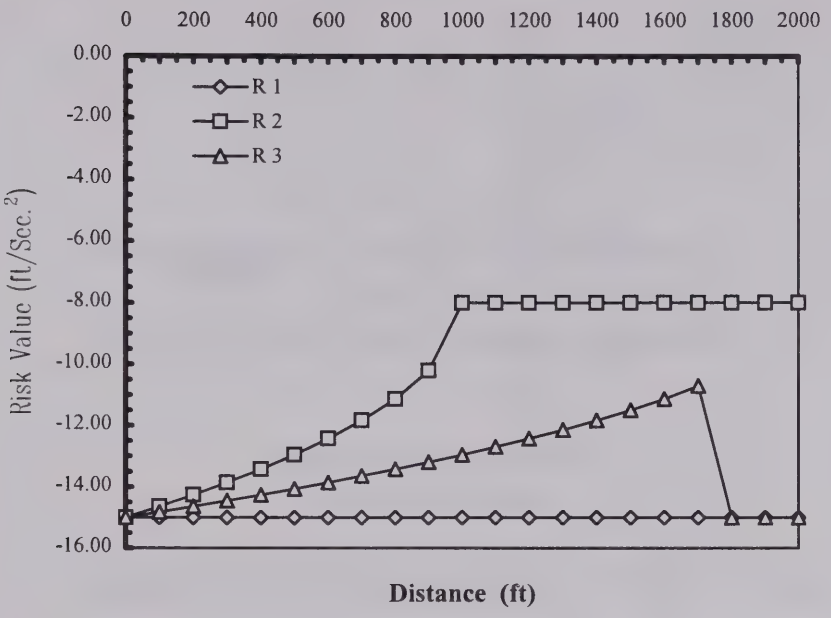


Figure 3-4 The Risk Value vs. Distance Between A Vehicle and An Off-Ramp



Unlike the risk value of a vehicle approaching an off-ramp, the risk values for a lane changer because of a incident warning sign or a lane drop is determined differently. The risk value decreases linearly from -8 ft/ sec^2 to -15 ft/sec^2 if the distance between the vehicle and the lane drop or the incident blockage increases. The risk value for a vehicle approaching a lane drop or an incident blockage is formulated as follows:

$$R = Z_{MN} + (Z_{MX} - Z_{MN})\left(\frac{d_3}{L_3}\right) \tag{3.9}$$

Where d_3 is the distance between the warning sign of the event (i.e., a lane drop or an incident blockage) and the vehicle, while L_3 represents the distance between the event and its warning sign. Figure 3-5a and 3-5b show the geometry for vehicles approaching an incident and a lane drop respectively, while Figure 3-6 presents how the risk values vary with the distance between the vehicle and the incident blockage. The results, shown in Figure 3-6, are contradictory to the real world situation because the drivers would take more risk to change lanes when the distance between the vehicle and the incident/lane-drop is reduced.

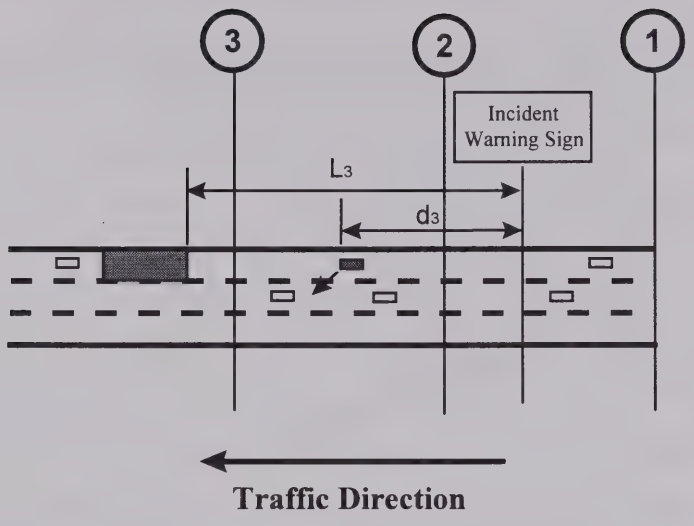


Figure 3-5a Vehicles Apporching An Incident Blockage

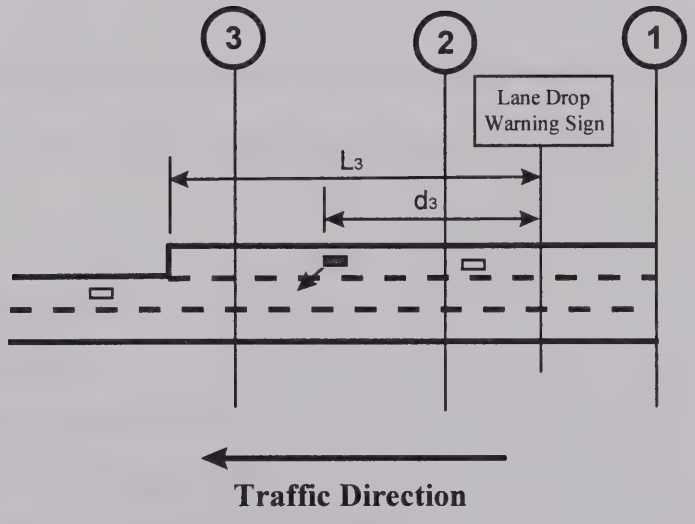
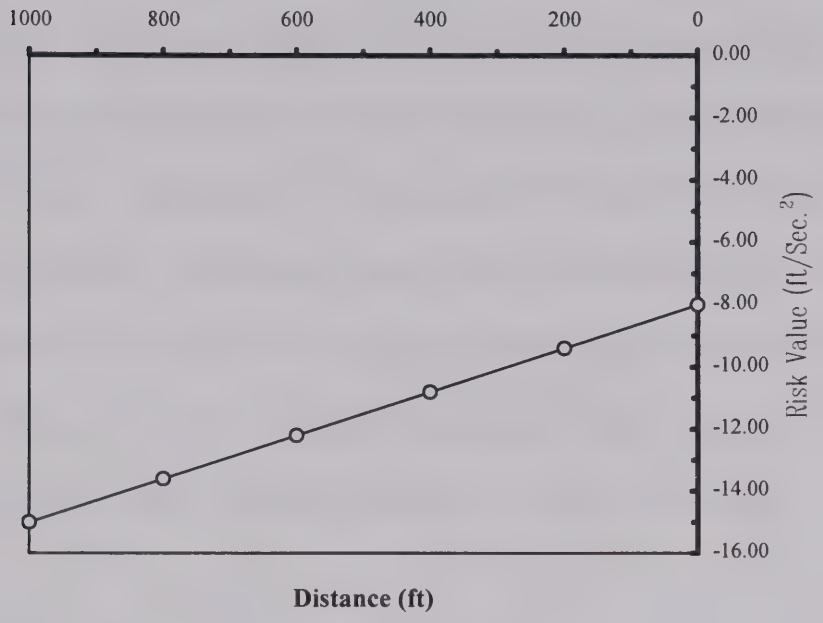


Figure 3-5b Vehicles Apporching A Lane Drop

Figure 3-6 Risk Value vs Distance Between A Vehicle and A Lane Drop/Incident Blockage



3.4 Other Pertinent Factors

The factors which affect freeway capacity and are not related to any type discussed previously, are identified and discussed in this section. These factors are :

- Rubbernecking factor
- Car following model
- Anticipatory lane change model
- Discretionary lane change model
- Structure of all driving attributes in FRESIM

3.4.1 Rubbernecking Factor

The term rubbernecking refers to the tendency of drivers to slow down as they pass an incident zone. This speed reduction results in lower throughput and therefore lower capacity. This rubbernecking factor causes additional capacity reduction, beyond that corresponding to the physical loss. Cragg and Demetsky [7] mention that the capacity reduction on a freeway found from previous studies, when one lane of a three lane facility was blocked by an incident, was appropriately 50%, rather than 33%. However, one can expect a minimum of 33% loss in physical capacity. Rubbernecking could also be used to model the effects, such as the traffic characteristics within work zones.

1. Current Logic:

The method to reduce lane capacity corresponding to a rubbernecking factor of a work/incident zone, is increasing the car-following sensitivity factor for every vehicle within the zone. Since the gap between a pair of vehicles increases, the lane capacity is reduced.

2. Disadvantage:

The reduced capacity within a work/incident zone is incurred by the gap between vehicle increases as well as the average speed of the zone decreases. Due to the limitation that all link free-flow speeds are fixed till the end of the simulation, users can only use the rubbernecking factor to calibrate the flow passing the zone. A comprehensive analysis for the speed and the density relationship within work/incident zones is suggested to be conducted in the future.

Another limitation in modeling incident zones in FRESIM is the specification of the rubbernecking factor. Currently, once one or more lanes are highlighted as opened lanes with impact from the rubbernecking factor, the impact is the same for all lanes. However, especially on a multi-lane freeway, the rubbernecking impact of the lane adjacent to the blockage is greater than the lane away from the blocked lane. For example, on a five-lane freeway, if lane 1 is blocked by an incident, the reduced capacity in lane 2 is greater than lane 5.

3.4.2 Car Following Model

1. Current Logic:

The pitt Car-Following Model is implemented in FRESIM to determine the proper acceleration rate of the follower vehicle. The resulting follower's acceleration rate is derived as:

$$a = \frac{2[X_{t+1} - Y_t - L_t - 10 - V_t(K_t + T) - bK(U_{t+1} - V_t)^2]}{[T^2 + 2KT]} \qquad (3.10)$$

where,

X_{t+1}: position of leader at time t+1

Y_t: position of follower at time t

L: length of the follower at time t

K: Car-following sensitivity factor of the follower

b: Calibration constant

U_{t+1}: speed of the leader at time t+1

T: a time step

V_t: speed of the follower at time t

Equation (3.10) represents the basic car-following relationship. The term involving the constant b is introduced to allow for a high relative closing speed behavior observed empirically. The value of b has been calibrated to

$$b = \begin{cases} 0.1 & \text{when} U_{t+1} - V_t \le 0 \\ 0 & \text{when} U_{t+1} - V_t > 0 \end{cases} \qquad (3\text{-}11)$$

2. Disadvantages:

Although it is important to understand that much of the programming logic in FRESIM is based on field data which in some instances may not have been as extensive as would have been desired. The constant b in the car-following model is one of the examples which should vary with the local driving behaviors as well as the level of congestion at the site. We suggest this value should be calibrated by users as they need.

3.4.3 Anticipatory Lane Change Model

1. Current Logic:

The current anticipated lane changing logic works as if the projected flow in a designated zone is higher than 1500 vphpl, the value of lane changing desire factor is automatically set to be 1. The designated zone length is the minimum of the upstream link length of the on-ramp gore or a constant 500 feet. There is no justified reason for selecting 500 feet as the designated zone length rather than other lengths. If the flow of that zone is less than 1500 vphpl, the comparison between the current vehicle speed v_c and the average speed in the designated area v_a is performed. The minimum value v_m selected between v_c and v_a is used to determine the desire factor D for a potential anticipatory lane changer. The driver's desire to make an anticipatory lane change is then computed from the following equations:

$$D = \begin{cases} 1 & \text{when} \quad v_m \le v_t \\ 1 - \dfrac{v_m - v_t}{v_d - v_t} & \text{when} \quad v_t < v_m < v_d \\ 0 & \text{when} \quad v_m \ge v_d \end{cases} \qquad (3.12)$$

where

D : the desire factor for an anticipatory lane changing

v_m : the minimum value of (v_c, v_a)

v_t : the vehicle tolerate speed, and

v_d: the vehicle desired free flow speed.

From Equation (3.12) the value of D ranges between 0 and 1. If D approaches 1 means 100 percent of vehicles will try to perform anticipatory lane changing. However, this fact does not imply that all vehicles can successfully change lanes because of the possible congestion in the adjacent lane.

2. Disadvantages:

(1) The first deficiency is the fixed reference flow rate 1500 vphpl in the designated zone, which is determined arbitrarily, and FRESIM does not provide calibrating feasibility to users. Furthermore, the designated zone size for calculating the referenced average speed downstream of the vehicle occupied lane is another deficiency. These values should not be constants and should vary with various factors, summarized in chapter 4.

(2) The anticipatory lane changing is not always dependent on the downstream traffic congestion. In most cases, vehicles performing anticipatory lane changing are based on other judgments, such as the speed of a vehicle merging from an on-ramp, shown in Figure 3-7b. In that figure, even the downstream traffic condition is not congested,

the vehicle i_1 will still make anticipatory lane changes because i_2 is trying to merge onto the mainline; although i_2 is still on the auxiliary lane.

(3) The warning sign associated with the downstream on-ramp congestion is automatically installed at 1,500 feet before the on-ramp gore. Unlike the distance between warning signs and objects, such as lane drops, off-ramps, and incident blockages, this distance can not be calibrated by users. However, the impact of capacity and speed reduction due to the distance between the warning signs and the on-ramp gore are yet unknown. More comprehensive analysis regarding the distance between the on-ramp gore and its warning sign are suggested to be conducted in the future.

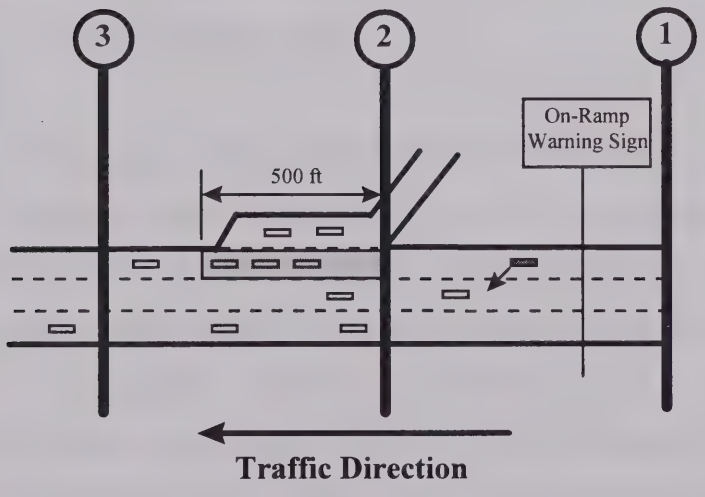


Figure 3-7a Existing Anticipatory Lane Chnages In An On-Ramp Geometry

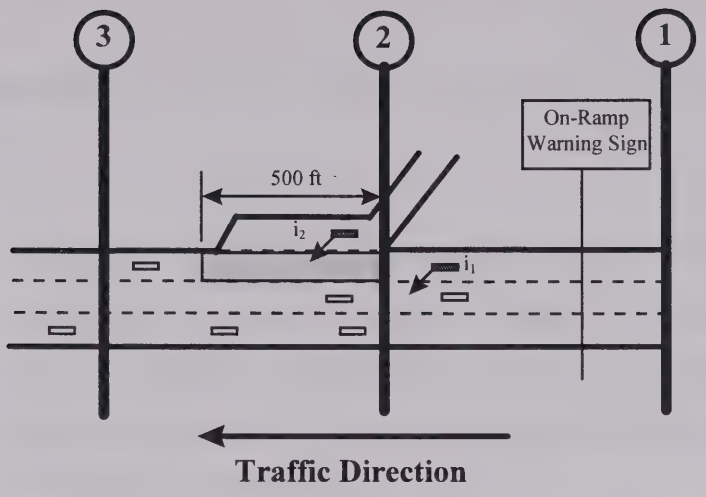
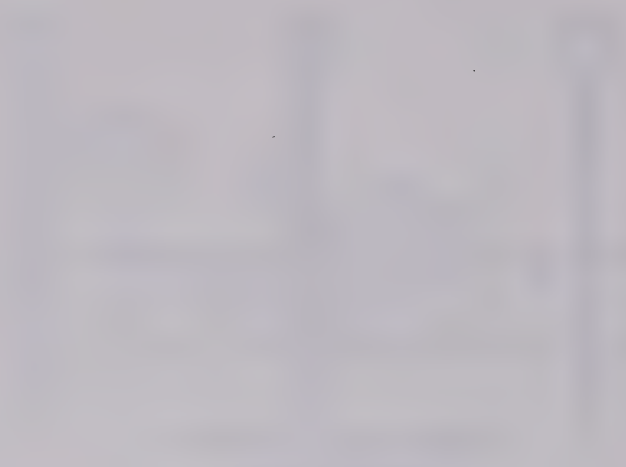


Figure 3-7b Suggested Anticipatory Lane Changes In An On-Ramp Geometry

3.4.4 Discretionary Lane Change Model

The current lane changing models in FRESIM include: (1) the anticipatory lane changing model, (2) the mandatory lane changing model, and (3) the discretionary lane changing model. The anticipatory lane changing refers to the downstream leader of congestion, while the mandatory lane changing refers to the warning signs associated with an off-ramp, incident, and a lane drop. In addition, the discretionary lane changing depends on Motivation, Advantage, and Urgency specified in the "Freeway Simulation Model Enhancement and Integration". Currently, "Motivation" models a vehicle's desire for a lane change. The model assumes that the degree of desire for the discretionary lane change is a function of the vehicle's current speed and the driver's characteristics. If the vehicle can not stand for the current operating speed, this vehicle will try to find a safe gap and shift to the adjacent lane.



1. Current Logic:

The current discretionary lane changing logic is performed by FRESIM as the following procedures:

Step 1 Identifying a vehicle whose current speed is less than it's desired speed

Step 2 Calculating the vehicle's desire to make a lane change based on the vehicle's desired speed, tolerate speed, and current speed.

Step 3 Checking whether the vehicle should be highlighted to be a potential discretionary lane changer.

Step 4 Identifying a corresponding gap on the adjacent lanes with respect to the current vehicle location.

Step 5 Calculating the risk value for the vehicle.

Step 6 Comparing the advantages between gaps on the adjacent lanes.

Step 7 Moving the vehicle to the safer gap if a safer gap is found. If both gaps are unsafe, the vehicle will stay in the current lane at the current time step, and the vehicle's urgency value is increased.

The vehicle urgency factor is adjusted based on the Equation. (3.13). The updated urgency factor for every time step (if the vehicle couldn't find a gap at this time step) is

$$I_{i,t} = I_{i,(t-1)} + \frac{(C_i + 1)}{20} \text{ where} \qquad (3.13)$$

$I_{i,t}$: the impatience factor of vehicle i at time t

C_i: the driver code of vehicle i

The incremented rate varies with driver type but the initial value of I_i is constant. From Equation. (3.12), the updated vehicle impatience value and risk value for various driver types at three consecutive time steps are summarized in Figures 3-8 and 3-9.

Figure 3-8 Urgency Value vs. Driver Type at Three Consecutive Time Steps (RLDFC=1)

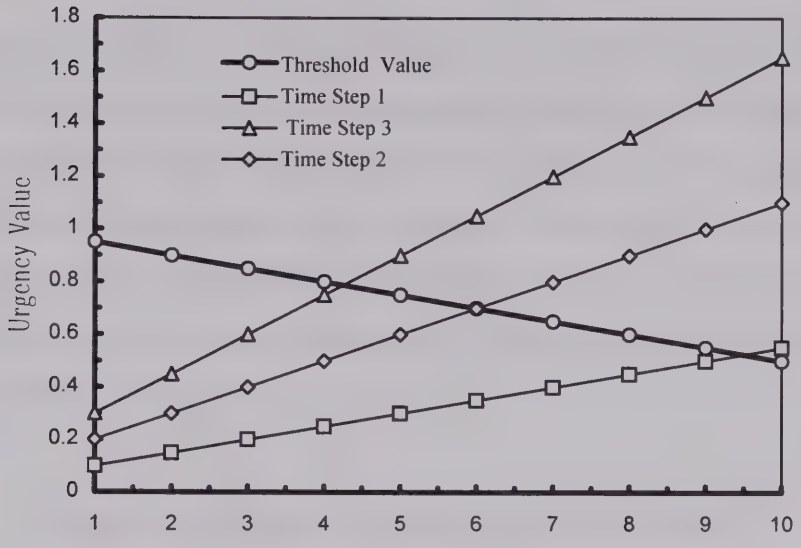
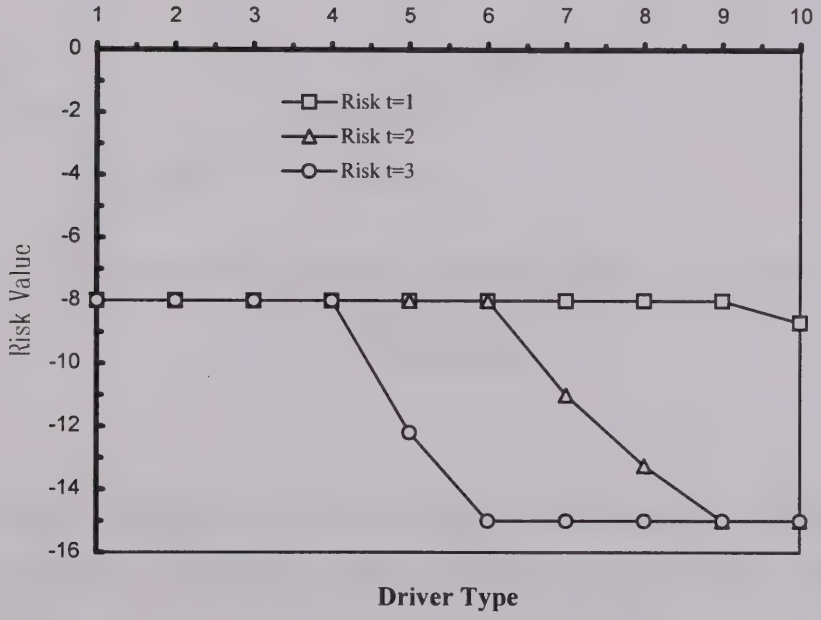


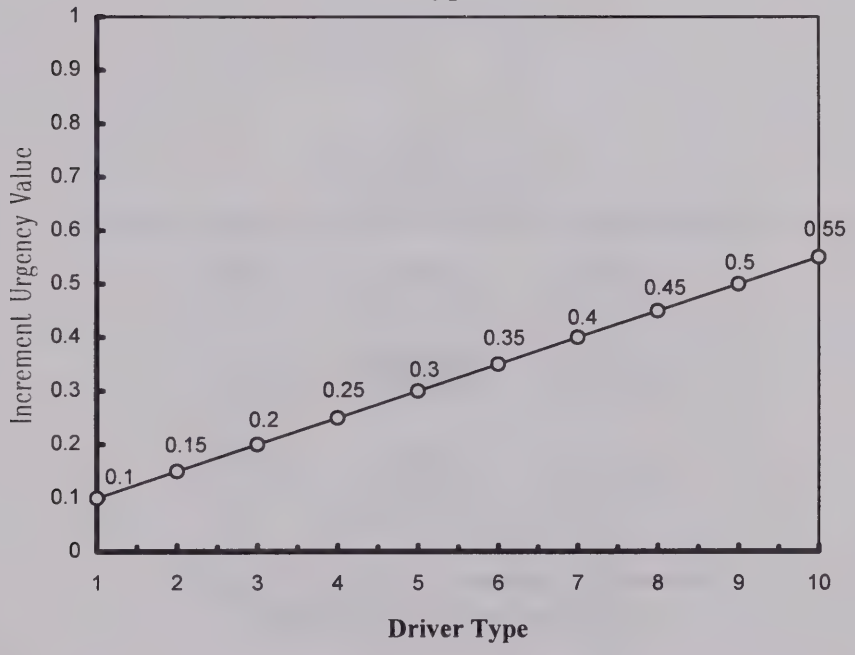
Figure 3-9 Risk Value vs. Driver Type at Three Consecutive Time Steps (RLDFC=1)



2. Disadvantages:

The threshold urgency factors are fixed and not modifiable by users in FRESIM. If the incremented value is big, the vehicle's urgency factor is very easy to exceed its threshold value. Thus the number of lane changers may increase. The main deficiencies in deriving the risk value for a potential lane changer include: (1) The fixed range of the threshold urgency values (i.e., from 0.5 to 0.95), and (2) The fixed increment urgency value for each driver type. The relationship between the increment value and the corresponding driver type is shown in Figure 3-10.

Figure 3-10 Urgency Increment Value vs. the Driver Type



Since the urgency value and the increment value do affect the number of lane changes as well as the capacity of freeway sections, we suggested that these values should be calibrated by users.

The current logic models only a portion of discretionary lane changers. Other factors affecting the motivations of vehicles to make lane changing are not considered in FRESIM, such as the motivation stimulated from its aggressive follower. Sometimes, the vehicle motivation is transmitted by the vehicles in adjacent lanes, such as drivers disliking parallel movements with trucks, particularly heavy and fully loaded trailers. The factors affecting discretionary lane changes should include the driver and vehicle types of the lead and follower vehicles, and the current lane that the vehicles occupy. From Figure 3-11, the target vehicle is IV and in general, vehicle IV's lane changing decision is motivated by the vehicles at various positions around IV. For example if IV_2 is a slow driver, IV will change lane, if IV_5 are trucks, IV will probably shift to the adjacent lane, if IV_7 is an aggressive truck, IV may shift to the right lane under pressures from its follower. Therefore, the current discretionary lane changing model is suggested to be enhanced.

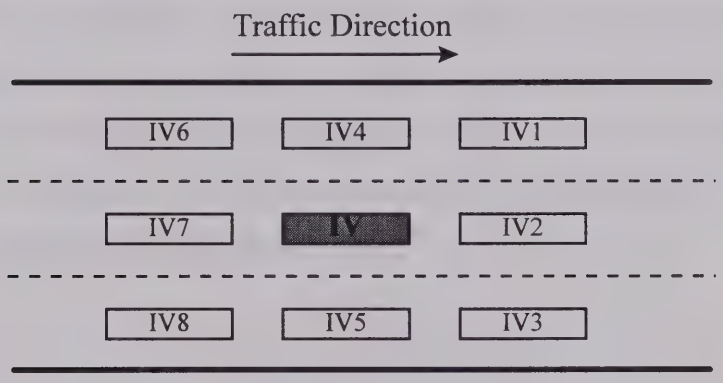


Figure 3-11 Lane Changing Motivation from Surrounded Vehicles

3.4.5 Structure of All Driving Attributes in FRESIM

1. Current Logic:

FRESIM defines ten driver codes from 1 through 10, based on vehicle desired speed. Values representing other attributes of a driver, such as the car following sensitivity factor, vehicle tolerate speed, and threshold value of urgency factor, are considered constant for a particular driver code. Which means all the attributes of drivers with the same driver code are the alike. Other driving attributes, such as reaction time and hiatus time are constant for all drivers.

2. Disadvantages:

Values of driver attributes should correspond to an individual's driving nature and ability. Therefore, instead of all attributes, one or more attributes of one driver may be the same as another driver. For example, a vehicle driver has the same desired speed with another driver but his/her other driving attributes, such as car following sensitivity factor, vehicle tolerate speed, and threshold value of urgency factor, may not match that of another driver's. Thus the structure of driver attributes considered in FRESIM could not represent actual driving attributes of an individual driver.

CHAPTER 4

RECOMMENDED IMPROVEMENTS ON FURTHER DEVELOPMENT OF FRESIM

The objective of this task is to provide suggestions to the FRESIM developer for developing a comprehensive microscopic freeway simulation model in order to obtain correct traffic information, such as the maximum flow passing a freeway section, called freeway capacity. According to the deficiencies indicated in Chapter 3, this chapter provides recommendations corresponding to each deficiency previously identified, which may improve the accuracy of the simulation results The enhanced FRESIM model should reduce the discrepancy between the simulation results and the data collected from the real world freeway network. This chapter is organized as follows:

- Recommendations in Driver Population Related Factors
- Recommendations in Vehicle Population Related Factors
- Recommendations in Freeway Geometry Related Factors
- Recommendations in Other Pertinent Factors

4.1 Driver Population Related Factors

In this section recommendations will be provided to all factors related to driver population whose deficiencies are identified in Chapter three.

4.1.1 Link Free Flow Speed

The link free flow speed is the average vehicle operating speed that could be achieved on a freeway link without any impedance from other vehicles and control devices. In light traffic conditions (i.e., level of service A), the speed difference between the speed that a vehicle can achieve on a link and the link free-flow speed, depends on the driver type (i.e., cautious or aggressive driver). Therefore, the driver code of a vehicle is one of the important factors that influence a vehicle's desired speed on a particular link. Other factors that affect the link free-flow speeds include, vehicle composition, the speed limit, and roadway conditions. No previous research was found that models the relationship between a vehicle's desired speed and the free-flow speed of the link where the driver occupies. However, the link posted speed limit does govern vehicle desire speeds [10, 15, 19], and the variable link speed limit will certainly affect driving behaviors, as well as the vehicle speed. Thus, our recommendations are:

1. The suggested vehicle desired speed should be time, link, and driver type dependent.

2. The posted speed limit should be an additional important factor to determine vehicle desired speeds in FRESIM.

3. The generated driver distribution should be improved, such as normal or gamma distribution, calibration, and ability to reflect the realistic patterns of driving behaviors.

4.1.2 Percentage of Cooperative and Non-Cooperative Drivers

According to our discussion in Chapter three, the risk value varies with the vehicle type and the percentage of vehicles that would yield their right of way. Therefore, our recommendations are proposed based on these and discussed as follows:

1. The risk values with respect to a vehicle meeting a putative follower bounded between values of Z_{MX} and $Z_{MN,}$ should be vehicle and driver types dependent, and calibratable by the users. The current design forces the percentages of cooperative drivers on all links as identical, which should be improved as link based specification.

2. Other factors, such as the level of congestion and the urgency of the lane changer, should affect the determination of Z_{MX} and $Z_{MN,}$ as well as the risk value. The example addressed in the Chapter three, regarding the work zone traffic operations, identified this deficiency.

3. The risk value assigned to the lane changer should be modified in subroutine FGAP.

4.1.3 Driver Distribution

Driving behaviors (i.e., the duration of the reaction lag time, the vehicle desired free flow speed, and the vehicle car following sensitivity factor) on a freeway are the combined outcome of one or more factors, such as the individual driving nature, the vehicle operational characteristics (i.e., acceleration and deceleration rates, and the control system), the roadway conditions (i.e., wet, dry, concrete, asphalt, surface roughness, lane width, and lateral clearance, etc.), and the weather condition (i.e., snow, fog, and rain). In addition, driving behavior may also vary with local land use characteristics and trip purposes. For example, a driver may be either a commuter when he/she travels between home and the work place; and a non-commuter when he/she enjoys his/her vacation in some recreational attractive places. Driving behaviors may be different because trip purposes are different. Thus, driver distribution should reflect the driver composition at the analyzed network. Therefore, we recommend that the driver distribution should be

calibrated, and the driving behavior should be varied with location, trip purpose (commuter and non-commuter), time period, and the level of congestion.

4.1.4 Vehicle Tolerate Speed

Every vehicle (driver) should have its tolerate speed dependent on the driver's characteristics (i.e., may be distinguished by driver types). Moreover, the vehicle tolerate speed may vary with traffic conditions. For example, in low density conditions, the tolerate speed of a vehicle may be higher than that in high density conditions. We recommend that the vehicle tolerate speed should be represented by a function of driver type and traffic conditions (i.e., congestion or non-congestion). Since the vehicle tolerate speed is determined by Equation (3-2), the parameters x and y should be calibrated corresponding to the local conditions.

4.1.5 Car-Following Sensitivity Factor

The car following sensitivity factor should vary with both driver and vehicle types. In addition, car following sensitivity factor further varies with roadway surface conditions, geometric configurations of roadway, weather conditions, and traffic conditions. According to the deficiencies identified in Chapter three, we recommend:

1. The car following sensitivity factor will be dependent on both driver and vehicle types.

2. The car following sensitivity factor will be adjusted internally with respect to the user specified lane width and lateral clearance of a link, roadway conditions, and weather conditions.

3. The link specific and time varied car-following sensitivity factors should be introduced in FRESIM for reflecting local driving behaviors.

4. The phenomenon that the car-following sensitivity factor varies with the level of traffic congestion should be validated in order to adjust the sensitivity factor rationally.

4.1.6 Driver Reaction Time

The reaction lag time should vary from one driver to another. Even one individual's driver reaction time may vary from one situation to another. For example, in congested conditions, drivers may be more cautious than in non-congested conditions. Driver reaction time also varies with weather conditions; if the driver is fatigued or alert, complexity of tasks, anticipated and unexpected situations. For instance, a driver's desire to accelerate for achieving her/his desired speed may take less perception time than deceleration; leaders that suddenly break and the followers need to decelerate to avoid collision, which may need more reaction time. Acceleration is always anticipatory, thus, acceleration lag time could be a constant value but the action corresponding to deceleration may be anticipatory or unexpected. Hence, deceleration lag time should be determined by a more complex form. More research is required to identify the nature of variation in reaction lag time among drivers. As reaction lag time has significant influence on freeway capacity, field observation of reaction lag time, and variation among drivers become important steps that need to performed. Our recommendations are summarized as follows:

1. Driver type, varied anticipatory, and expected lag time should be introduced into the model.

2. The driver response lag time should not be a constant for all driver types. The recommended way for determining the lag time is first identifying all factors affecting the duration of the response time. The driver response lag time is

dependent on the driving behavior in terms of driver distribution, the level of traffic congestion, the roadway geometry, and the acceleration or deceleration rate of the vehicle. The default response lag times for acceleration (ZLAGAC) and deceleration (ZLAGDC) can still be implemented in the FRESIM, but the duration of the lag time for each vehicle should be adjusted based on Equation (4-1).

$$RLAG = f(x_1, x_2, x_3, x_4, x_5,) \quad (4\text{-}1)$$

where

RLAG means either the response lag time for acceleration or deceleration,

x_1 : the driver type of the subject vehicle,

x_2 : the level of congestion,

x_3 : the driver code of the subject vehicle,

x_4 : the acceleration or deceleration rate of the subject vehicle, and

x_5 : the code of geometry.

Constraints should be subjected to the model, such as RLAG ≤ ZLAGDC when performing deceleration, and RLAG ≤ ZLAGAC when performing acceleration.

3. The driver response lag time should not be implemented by the relative acceleration or deceleration rates for the current and the previous time steps. If acceleration or deceleration of a vehicle is continuous from the previous time step, the driver response lag time should not be taken into account in either the car following model or the anti-collision model.

4.1.7 Hiatus Time Period

According to the hiatus period issue discussed in Chapter three we will provide several alternatives to alleviate the abnormal congestion on freeway networks.

1. Adjust the duration of the hiatus period which was discussed yesterday.

2. If "1" does not work, calibrating the non-emergency deceleration rates for all vehicle types in Card Type 70. The suggested rate should start from 9 until 15 ft/sec^2 and increase 1 foot per second square, in every consecutive adjustment for every vehicle type. This calibration may stop until the results are satisfactory, after comparing the simulation results and the MOEs are observed from a real traffic situation.

3. If "2" fails, assign a temporary duration of RTA in subroutine FGAP based on some potential factors affecting RTA. (This alternative is just temporarily designed to represent the proper hiatus period for various driver types at different conditions. FHWA should find a theoretical way to overcome this problem in the future.)

   Assuming "v" represents a vehicle's identification number, the duration of the hiatus period is dependent on the factors including :

   (a) the driver type of the subject vehicle (the lane changer), named D_V
   (b) the fleet type of the subjective vehicle, named F_V
       where, F_V = (0,1,2,3) representing (passenger cars, trucks, carpools, buses)
   (c) the urgency factor of the subject vehicle v
   (d) the driving behavior of the site, and
   (e) the level of congestion in the site

   The impact of level of congestion to the duration of hiatus period is not considered in this discussion because it involves more assumptions on this issue. In addition, the urgency factor is highly dependent on the driver code of the lane changer [4]. Therefore, the duration of the hiatus period of the

subjective vehicle (h_v) is formulated based on the driver code and the vehicle type of the subjective vehicle. The logic which can be implemented into FRESIM for providing the proper duration of the hiatus period for each vehicle is programmed as follows:

IF ($F_v = 0$ or $F_v = 3$) THEN

  $h_v = H/(D_v)^x$

ELSE IF

  $h_v = H$

END IF

Where H means the default duration of the hiatus period (say 3 seconds) and x represents the index of driving behavior patterns at the site. The range of x is suggested from 0 to 0.5, because of the consideration for the minimum time duration to complete a lane change.

The rationale for the designed provisional model is based on:

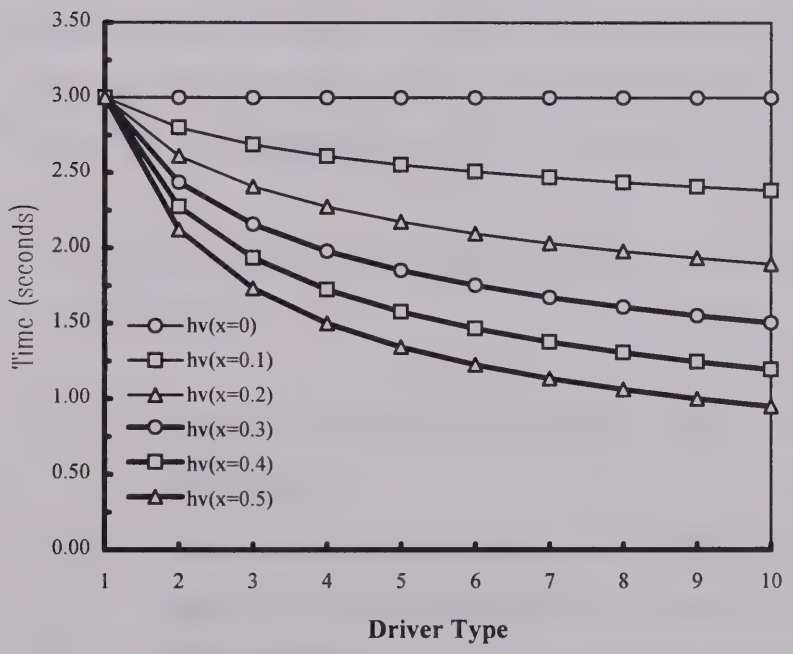
(a) Passenger cars and carpools have better operational maneuver ability than heavy vehicles. Therefore, the default (longer) duration of hiatus period will be assigned to all heavy vehicles.
(b) The aggressive driver tends to have less time spent for changing lanes.
(c) The duration of hiatus period for general passenger cars and carpools is determined based on the local driving behaviors of the site.

The suggested hiatus period for each vehicle at various conditions is summarized in Table 4-1 and Figure 4-1.

Table 4-1 Suggested Hiatus Period vs. Driver Type at Different Indexes of Driving Behaviors

D_V	$h_V(x=0)$	$h_V(x=0.1)$	$h_V(x=0.2)$	$h_V(x=0.3)$	$h_V x=0.4)$	$h_V(x=0.5)$
1	3.00	3.00	3.00	3.00	3.00	3.00
2	3.00	2.80	2.61	2.44	2.27	2.12
3	3.00	2.69	2.41	2.16	1.93	1.73
4	3.00	2.61	2.27	1.98	1.72	1.50
5	3.00	2.55	2.17	1.85	1.58	1.34
6	3.00	2.51	2.10	1.75	1.47	1.22
7	3.00	2.47	2.03	1.67	1.38	1.13
8	3.00	2.44	1.98	1.61	1.31	1.06
9	3.00	2.41	1.93	1.55	1.25	1.00
10	3.00	2.38	1.89	1.50	1.19	0.95

Figure 4-1 Hiatus Period vs. Driver Type at Various Indexes of Driving Behaviors



From Figure 4-1, the average duration hiatus period decreases if the index of driving behavior increases. The minimum hv is assumed to be 0.95 seconds (see Table 4-1 when $D_v = 10$ and $x = 0.5$). Any other values lower than 0.95 seconds is impossible to achieve a complete lane change.

4.1.8 Warning Signs

In reality, as regular commuters are familiar with the location of their designated off-ramps, they may change to their proper lanes prior to the warning sign. Drivers who are not familiar with the location of warning signs, will change lanes after passing the sign. Our recommendations are discussed as follows:

1. Traffic conditions and driver types based the probabilistic mandatory lane changing model in relation with the distance between the warning sign and the associated object (i.e., off-ramp, lane drop, or blockages). This should be introduced, which means the number of lane changes will distribute between warning signs and the objects at a statistic distribution.

2. Currently FRESIM should be enhanced for allowing multiple warning signs associated with an object. This modification can represent more complex distribution other than uniform, normal, or gamma distribution.

3. Our suggested mandatory lane changing logic for off-ramp warning signs is described below:

   Step 1 Identify a designated zone bounded by a fictitious warning sign and an off-ramp.

   Step 2 Divide designated zone into some suitable subzone, with equal length. Set a vehicles desirability to change lanes at the beginning of each subzone.

Step 3 Identify an off-ramp vehicle that is not in the candidate lane at the time crossing the fictitious warning sign, and determine that vehicle's desire to change lanes at the beginning of a particular subzone; from the probability distribution of lane changing desirability at the beginning of each sub zone.

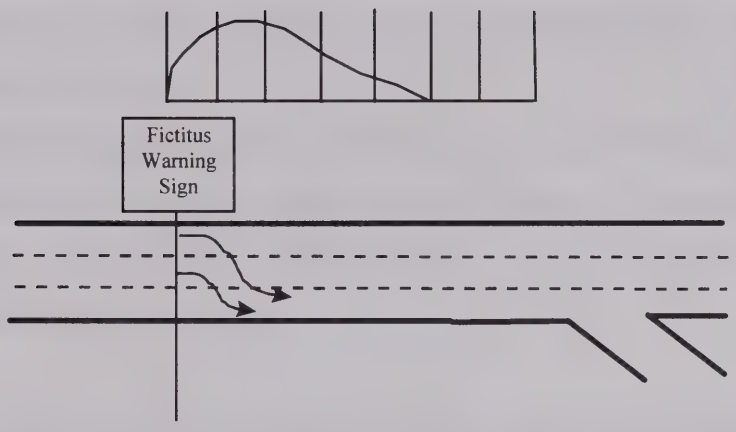


Figure 4-2 Suggested Distribution of Mandatory Lane Change

4.1.9 Driver Lane Preference Factor

In light traffic conditions vehicles are distributed along freeway links in such a way that vehicles have little or no difficulty maintaining there desired speed. In these traffic conditions vehicles may choose lanes based on their desired speed. Usually, higher speed vehicles choose the left most lane, and slower vehicles gradually shift toward right lanes. Thus, in light traffic conditions each lane is used by vehicles with a range of desired speeds. This situation is not handled by the current FRESIM model; and to simulate this situation, FRESIM should consider lane preference factors for each vehicle, based on desired speeds.

To introduce the lane performance factor in FRESIM we recommend that each vehicle selects an "appropriate" lane, based on its desired speed and the lane preference factor. The suggested model is discussed as follows:

Step 1 From field observations, identify vehicle distribution and the range of vehicles' desired speeds in each lane.

Step 2 Identify ranges of vehicles' desired speeds that are only observed in a particular lane and set those lanes as preference lanes for those vehicles suitable for the lanes.

Step 3 Identify ranges of vehicles' desired speed that are observed in more than one freeway lane. Distribute each of those vehicles to any one of the lanes in which the vehicle's desired speed is observed based on the vehicle distribution in each lane.

If a vehicle's current position is not in its preferred lane, even if it maintains its desired speed in the current lane (e.g., a slow vehicle in the left most lane), the vehicle will try to shift to its preferred lane if it is able to maintain its desired speed in the preferred lane by changing lanes. The suggested discretionary lane changing model, discussed in section 4.4.4, will handle such lane changing behaviors. Lane preference speed set in Step 2 should be calibrated by users.

4.1.10 Threshold Value of the Urgency Factor

A common threshold value of the urgency factor for all drivers under a particular driver type is not realistic. Our recommendations include:

1. The threshold urgency factor should be calibrated by users because it is highly dependent on the types of drivers. Therefore, the threshold urgency factor should be fine tuned after the driver distribution is calibrated.

2. Additional impact analysis for the threshold urgency factor on traffic conditions is necessary, such as (a) urgency factor vs. flow, (b) urgency factor vs. Speed, (c) urgency vs. # of lane changes (d) urgency factor vs. delay.

4.2 Vehicle Population Related Factors

In this section recommendations will be provided for all factors related to vehicle population related factors, whose deficiencies are identified in Chapter three.

4.2.1 Percentages of Fleet Components

Our recommendations for enhancing fleet components in FRESIM are:

1. The comparative analysis for the traffic impact, which was observed from simulation results and data was collected from the field sites, for various combinations of vehicle types to the freeway capacity, average speed, and delays should be conducted in the future.

2. The distribution of vehicle types in each fleet should be feasibly calibrated by users. The percentages of the vehicle types should vary with different time periods and entry links.

3. Bus length and bus acceleration/deceleration rates should be calibrated by users, so that users can simulate realistic bus operations on bus routes, because they affect bus travel time. The additional bus types should be introduced into FRESIM and are suggested in Table 4-2.

Table 4-2 New Vehicle Types for Buses

Fleet	Vehicle Type	Length
Buses	1 Minibuses	20 ft
	2 Standard Buses	40 ft
	3 High Capacity Buses	60 ft

4.2.2 Truck Turn Movements (Specific Truck O-D Matrix)

The current vehicle turn movement logic should remain in the FRESIM logic and should be utilized in the analysis of design and planning work when users can not collect the actual truck O/D demand information. However, for improving the accuracy of the simulation output in freeway operational analysis by using FRESIM, the following modifications are suggested:

1. In card type 25, the specification of vehicle turn movement percentage is suggested to be divided into three parts, such as the percentages of passenger cars, carpools, and trucks.

2. In card type 74, the input regarding vehicle O/D information may follow the same structure specified previously and be replaced by various O/Ds categorized by vehicular types.

4.2.3 Minimum Separation for Generation of Vehicles

FRESIM generates vehicles with the concept of minimum separation of generation of vehicles (e.g., the default value is 2 seconds.), and the vehicles are emitted uniformly or according to user specified lane distribution over the lanes of the entry link. The major problem that exists in the FRESIM model for generating vehicles is that the minimum

separation in generating vehicles is common for all entry links; regardless if they are congested or not. To overcome the problem, we recommend the following alternatives:

1. The minimum separation between the emitted vehicle and its immediate leader should be consistent with the car following sensitivity factor of the network.

2. The vehicle emission interval should follow a distribution that will fit with real world freeway traffic distribution.

4.2.4 Vehicle Maximum Deceleration Rate

The vehicle maximum deceleration rate varies with vehicle types and is constant for a particular vehicle. To reflect maximum deceleration rate based on vehicle performance we recommend the following.

1. The vehicle emergency deceleration rate should not be common for all vehicle types. The suggested design is following the non-emergency deceleration, which can be calibrated by users for every vehicle type.

2. There is a possibility that the adjusted emergency deceleration rate (Z_{MX}) is less than the adjusted non-emergency deceleration rate if users are not aware of it, these values are specified in different card types (e.g., 70 and 71). The suggestion is to check the values of Z_{MN} and Z_{MX}. If $Z_{MN} < Z_{MN}$, without which the simulation should not be performed.

3. The jerk values of vehicle are suggested to vary with driver type, vehicle type, and grades. Not every driver uses the maximum jerk to achieve his/her desired speed.

4.2.5 Mean Start-Up Delay

Our recommendations regarding the enhancement for the determination of vehicle start-up delay are discussed as follows:

1. The test of the start-up delay impact should be conducted before further modifications. Related literature regarding the start-up delays distribution respect to driving behaviors should be reviewed.

2. The suggested enhancement is deriving a start-up delay model. This model should be a function of driver types, vehicle types, traffic conditions and may be the type of ramp metering control.

4.2.6 Truck Movements Before Work Zones or Incident Areas

We suggest to enhance the current mandatory lane changing logic for work zones or incident areas so that unusual truck delays are eliminated.

4.3 Freeway Geometry Related Factors

In this section recommendations will be provided to all factors related to freeway geometry, whose deficiencies are identified in Chapter three.

4.3.1 Weather Conditions

Weather conditions, such as fog, snow, and rain, affect freeway capacity in varying severity, depending on how bad the conditions. One of the major factors, such as rain, affects capacity differently based on its intensity.

Two factors that are involved in reducing freeway capacity because of bad weather are:

(1) reduction in visibility, and

(2) reduction of friction factor between the road surface and vehicle tires.

Roadway surface conditions deteriorate because of the bad weather which reduces the vehicle controlling performance. Thus, the car following sensitivity factor increases and the vehicle speeds are reduced. Reduction in speed and increase of the car following factor, depends on the severity of bad weather. Therefore, the car following sensitivity factors can be designed as link based, in order to reflect bad weather conditions on some specific links. Further research needs to be conducted; such as determining speed variations and speed reduction rates in different weather conditions.

4.3.2 Lane Width and Lateral Clearance

The car following sensitivity factors increase with the reduction of lane width and lateral clearance. There is no established relationship between car following sensitivity factors and lane width and lateral clearance. We suspect that with the reduction of lateral clearance and lane width, average speed may also be reduced, in comparison to standard lane width and lateral clearance in similar traffic conditions. More research is essential in this regard. We found that with the reduction of lateral clearance and lane width the service flow rate will be reduced [17]; therefore, the capacity will be reduced. Table 4.3 provides factors for restricted lane width and lateral clearance to estimate the service flow rate and thus to estimate freeway capacity. Table 4.3 should be incorporated in FRESIM to reflect similar change in capacity, according to user specified lane width and lateral clearance, by varying car following sensitivity factors.

TABLE 4-3 Adjustment Factor For Restricted Lane Width and Lateral Clearance

DISTANCE FROM TRAVELED WAY TO OBSTRUCTION[a]	ADJUSTMENT FACTOR					
	OBSTRUCTION ON ONE SIDE			OBSTRUCTION ON TWO SIDES		
	LANE WIDTH[a].(FT).					
(FT)	≥12	11	10	≥12	11	10
≥6	1.00	0.95	0.90	1.00	0.95	0.90
4	0.99	0.94	0.89	0.98	0.93	0.88
2	0.97	0.92	0.88	0.95	0.90	0.86
0	0.92	0.88	0.84	0.86	0.82	0.78

[a].Interpolation may be used for lane width or distance from traveled way to obstruction.
Source: 1994 Highway Capacity Manual

4.3.3 Warning Signs for Grade Geometry

As FRESIM emits vehicles in any lane irrespective of their desired speed, slow vehicles, such as trucks, may still operate on the left most lane. Therefore, unnecessary lane changing phenomena were observed from TRAFVU. This situation becomes worse when slow vehicles operate at grade terrain as their speeds are further reduced. Due to the operational characteristics of slow vehicles, they can not maintain their desired speed at grade terrain. Hence, we introduce a new warning sign category to serve grade geometry situations.

4.3.4 Risk Factor

Our recommendations are summarized as follows:

1. More comprehensive tests regarding risk factors are suggested to be conducted. The recommended relationships between the distances of (vehicle, off-ramp) and (off-ramp, off-ramp warning sign) are shown in the following figures. Figure 4-3 shows that the risk value is low when a vehicle just passes the warning sign within 200 feet. Afterwards, the risk value is non-linearly

decreased, while the vehicle approaches the off-ramp. If the vehicle reaches the off-ramp, the minimum risk value yields -15. Figure 4-4 shows a different scenario for determining the risk value to vehicles. If a vehicle passes a warning sign, its risk value, like the previous scenario, decreases non-linearly. However, when the vehicle is in the zone (say 200 feet to the off-ramp), the risk value yields the minimum -15. This suggestion can increase the probability of vehicles successfully changing lanes mandatorily, when vehicles are destined to an off-ramp.

Figure 4-3 The Risk Value vs. The Distance Between A Vehicle and An Off-Ramp

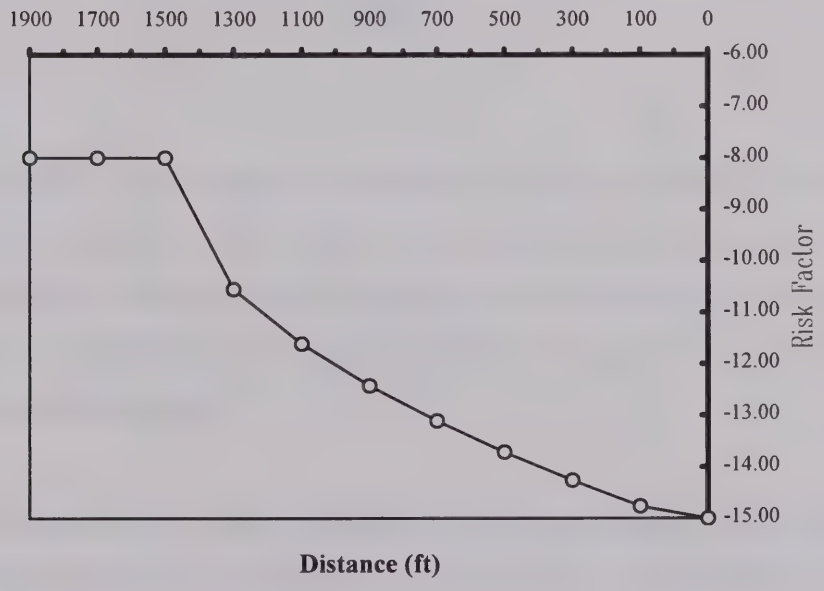
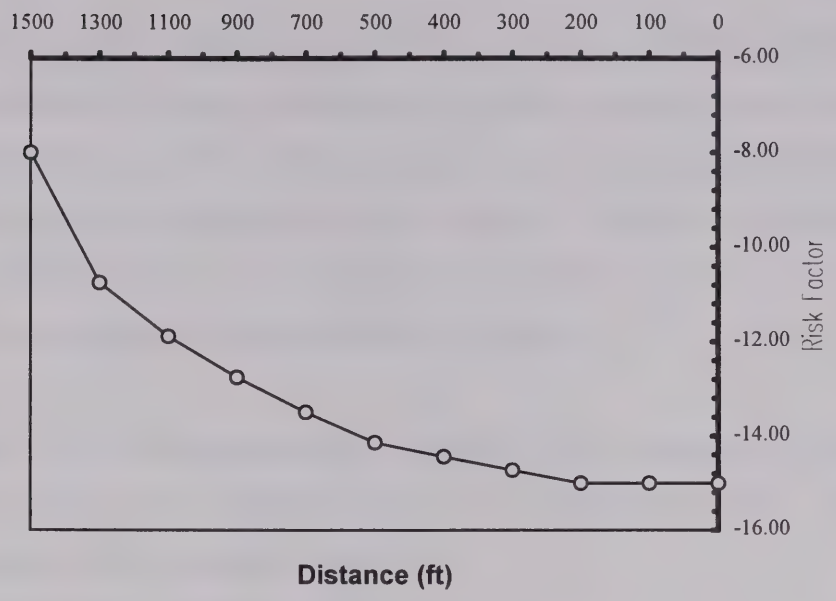


Figure 4-4 The Risk Value vs. The Distance Between A Vehicle and An Off-Ramp



2. The vehicle lane changing distribution should be carefully observed. For various locations, driver types, and vehicle types, the lane changing distributions, over the distance between the warning sign and the off-ramp, must be very different. Guidelines should be developed for instructing users to calibrate their models.

3. The determination of risk factors for vehicles approaching a lane drop and/or incident blockage may follow the same logic recommended for vehicles approaching off-ramps, if the non-linear model works well.

4.4 Other Pertinent Factors

In this section recommendations will be provided for all factors whose deficiencies are identified and discussed under other factors in Chapter three.

4.4.1 Rubbernecking factor

Due to the freeway capacity reduction in incident or work zones, queue may be formed at upstream of such an area. At the starting point of reduced capacity area, vehicles are discharged from the queue. Queue formation and queue discharging depend on the vehicle arrival rate and the amount of capacity reduction. However, the rubbernecking factor for the opened lanes in a designated incident area should not be common. Therefore, the queue lengths for all opened lanes are different.

We recommend that the rubbernecking factor should be time dependent, link and lane specific, in reflecting realistic traffic operations in incident/work areas. Additional analysis is suggested to be conducted, such as:

(a) Capacity reduction vs. Rubbernecking factor

(b) The density and speed analysis with/without an incident area.

4.4.2 Car Following Model

Our recommendations are:

1. The parameter b in Equation 3.6 should be opened for user's calibration.

2. The 10 feet safety distance should be determined based on the local driving behaviors and traffic conditions. In congested conditions, the distance is usually less than 10 feet.

3. Other car-following models such as the General Motors model are suggested to be studied in order to find the potential deficiencies in the Pitt car-following model.

4.4.3 Anticipatory Lane Change Model

In freeways, especially in proximity to on-ramp and off-ramp areas, vehicles changing lanes can not be fully modeled, by mandatory or discretionary lane changing models, because of various motivations. FRESIM simulates such lane changing phenomena by the module called anticipatory lane changing module. Our recommendations related to this issue are summarized as follows:

1. Distance of warning sign associated with the downstream on ramp congestion calibrated by user instead of using a fixed number 1500 ft. Additional analysis is necessary to be done for evaluating the anticipatory lane changing model:

    (a) Distance between an on-ramp gore and its warning sign vs. capacity (maximum observed flow)
    (b) Distance between an on-ramp gore and its warning sign vs. average link speed
    (c) Distance between an on-ramp gore and its warning sign vs. number of lane changes
    (d) Distance between an on-ramp gore and its warning sign vs. various roadway geometries

2. Designated zone length should be a variable for calculating the reference average speed. This should vary with local geometric conditions or allow users to calibrate it.

3. The variable LSPCMN represents the space mean speed in subroutine ANTCP, has never been used, and it is suggested to be removed from embed1.for

4.4.4 Discretionary Lane Change Model

FRESIM currently can not efficiently handle discretionary lane changes, especially for slow vehicles that are operating on the left most lane. A slow vehicle will not move toward right lanes if it is able to maintain its desired speed or if there is no mandatory lane change required. Thus, the slow vehicle may force its follower to change lanes if the desired speed of its follower is higher than the current speed of the slow vehicle. But in reality slow vehicles will shift towards the right lane. To overcome this problem we recommend the following discretionary lane changing logic:

Step 1 Identify a vehicle's speed in this time step. If the vehicle's speed is less than its desired speed go to step 2, otherwise go to step 3.

Step 2 Calculate the vehicle's desire to make a lane change based on the vehicle's desired speed, tolerate speed, and current speed and shift the vehicle to the proper lane as soon as a safe gap is found.

Step 3 Identify the vehicle's location in this time step. If the vehicle is in its preferred lane, go to step 4; otherwise, go to step 5.

Step 4 Keep the vehicle in the current lane.

Step 5 Check the speed of the adjacent lane towards the vehicles preferred lane. If the speed of the adjacent lane is less than its desired speed, keep the vehicle in the current lane; otherwise shift the vehicle to the adjacent lane as soon as safe gap is found.

The suggested discretionary lane changing model will help slow vehicles that are emitted onto a left most lane through entry link to shift towards right lane.

It may be possible to avoid the lane preference factor in the discretionary lane changing model, if more efficient discretionary lane changing models are developed that will be sensitive to the following factors:

$VDLS = f(x_1, x_2, x_3, x_4. x_5,)$ (4-2)

where,

VDLS = Vehicles discretionary lane change, i.e., (0,1) = (Yes, No) to perform Discretionary Lane Change,

x_1 = Vehicles desired speed,

x_2 = Vehicles current speed,

x_3 = Vehicles tolerate speed,

x_4 = Speed on left adjacent lane (if any), and

x_5 = Speed on right adjacent lane (if any).

4.4.5 Structure of All Driving Attributes in FRESIM

For more accurate approximation of drivers' driving attributes, that are consistent with reality, we recommend that for a given desired speed, a range of values representing each driving attribute based on field observation should be provided by the user. Then, an individual driver's driving attributes, will be determined from the range of values of each attribute stochastically.

CHAPTER 5

SUMMARY

The observed maximum flow rate from the simulation results can be viewed as the approximation of freeway capacity. It is a major component in the calibration of MOEs [17] which determine levels of service on freeway networks.

This project reviewed the background literature regarding capacity analysis models to identify limitations and deficiencies in the microscopic freeway simulation model (FRESIM). Specific problems that were addressed in Chapter three were classified into four parts, including factors affecting freeway capacity analysis; such as driver, vehicle, and geometry, related characteristics. Other factors such as rubber necking, car following factor, anticipatory lane change model, discretionary lane change model, and structure of all driving attributes in FRESIM were discussed in the section 3.4. The recommendations and suggestions for each identified deficiency were summarized in chapter four.

Deficiencies of various factors in the current FRESIM model are identified by comparing the logic of each factor in FRESIM and its validity on the basis of the findings from

various related literature. Due to lack of extensive research in some of the areas, concrete recommendations were not capable of being provided. This research helped to identify areas that need further study.

REFERENCES

1. Bacon, V. W. Jr., et al., "Use of INTEGRATION Model to Study High-Occupancy-Vehicle Facilities," Transportation Research Record No. 1446, Transportation Research Board, Washington, D. C., 1994.

2. Blumenfeld E. Dennis and Weiss H. George, "Merging from an Acceleration lane", Transportation Science, Vol. 5, No.2, May 1971, pp. 161-168.

3. Brilon W. and M. Ponzlet, "Variability of Speed-Flow Relationships on German Autobahns", Transportation Research Record No. 1555, Transportation Research Board, 1997, pp. 91-97.

4. Chien, I-Jy Steven and Lieu, C. Henry, " Enhancement in Generating Driver Population for The Microscopic Freeway Simulation Model (FRESIM)," Working Paper 97-1-2, New Jersey Institute of Technology, August 1997.

5. Cohen, S. and Clark, J., "Analysis of Freeway Reconstruction Alternatives Using Traffic Simulation," Transportation Research Record No. 1132, Transportation Research Board, Washington, D. C., 1986, pp. 8-13.

6. "CORSIM User Manual", Federal Highway Administration, U.S. Department of Transportation, Contact No. DTFH61-92-Z-00074, 1996.

7. Cragg, A. Catherine and Demetsky, J. Michael, "Simulation Analysis of Route Diversion Strategies for Freeway Incident Management" Virginia Transportation Research Council, Charlottesville, Virginia, VTRC 95-R11, January 1995.

8. Fazio, J. and Rouphail, N., "Conflict Simulation in INTRAS: Application to Weaving Area Capacity Analysis," Transportation Research Record No. 1287, Transportation Research Board, Washington, D. C., 1990, pp. 96-107.

9. FHWA, "Passenger Car Equivalents for Rural Highways", FHWA/RD-82/132, Federal Highway Administration, Washington, D. C., 1982.

10. Freedman, Mark and Williams, F. Allan, "Speeds Associated with 55 mph and 65 mph Speed Limits in Northeastern States". ITE Journal, February 1992, pp 17-21.

11. Garber, Nicholas J. and Gadiraju, Ravi, "Factors Affecting Speed Variance and Its Influence on Accidents," Transportation Research Record No. 1213, 1988, pp 64-71.

12. Halati, A. and Torres, J., "Freeway Simulation Model Enhancement and Integration - FRESIM User's Guide," Federal Highway Administration, Report No. DTFH61-85-C-00094, 1990.

13. Halati, A. and Torres, J., "Freeway Simulation Model Enhancement and Integration - FRESIM Technical Report," Federal Highway Administration, Report No. DTFH61-85-C-00094, 1990.

14. Halati, A. and Torres, J., "Freeway Simulation Model Enhancement and Integration - FRESIM Calibration/Validation," Federal Highway Administration, Report No. DTFH61-85-C-00094, 1990.

15. Harkey, L. David, Robertson, H. Douglas, and Davis, Scott E., " Assessment of Current Speed Zoning Criteria". Transportation Research Record 1281, Transportation Research Board, Washington, D.C., 1990, pp 40-51.

16. Hellinga, B. R. and Shortreed, J. H., "FREEVU: A Computerized Freeway Traffic Analysis Tool, " Transportation Research Record No. 1360, Transportation Research Board, Washington, D. C., 1992, pp. 30-36.

17. "Highway Capacity Manual, Special Report 209," Transportation Research Board, Washington, D. C., 1985.

18. "Highway Statistics 1995," FHWA, US Department of Transportation, Washington, D. C., 1995.

19. Jernigan, D. J. and Lynn, W. C., " Impact of 65-mph speed limit on Virginia's Rural Interstate Highways through 1989" Transportation Research Record No. 1318, 1991, pp 14-21.

20. Jones, E. R. and Goolsby, M. E., "The Environmental Influence of Rain on Freeway Capacity". Highway Research Record 321, Transportation Research Board, Washington, D.C., 1970, pp. 74-82.

21. Leiman, L., et al., "Integrated System of Freeway Corridor Simulation Models," Transportation Research Record No. 1320, Transportation Research Board, Washington, D. C., 1991.

22. Lindley, J. A., "Urban Freeway Congestion Problems and Solutions: An Update," ITE Journal, Vol. 59, No. 12, Institute of Transportation Engineers, Washington, D. C., 1989, pp. 21-23.

23. Lu, J., Huang, W., and Mierzejewski, E. A., "Driver Population Factors in Freeway Capacity", Report No. 0510759, Florida DOT, 1997.

24. May, A. D., Traffic Flow Fundamentals, Prentice-Hall Inc., Englewood Cliffs, Ney Jersey, 1990.

25. McShane, W. R. and Roess, R. P., "Traffic Engineering," Prentice Hall, Englewood Cliffs, New Jersey 07632, 1990.

26. Michaels M. R. and Fazio J., "Driver Behavior Model of Merging", Transportation Research Record No. 1213, Transportation Research Board,1989, pp. 4-10.

27. Mingo, R. D. and Zhuang, L., "Passenger Car Equivalents of Large Trucks, Derived from Use of FRESIM Model," Draft Paper submitted for publication to TRB, July 1994.

28. NCHRP 3-45, Speed-Flow Relationships for Basic Freeway Segments, Transportation Research Board, May, 1995.

29. Nemeth, Z. A. and Rathi, A. K., "Potiential Impact of Speed Reduction at Freeway Lane Closures: A simulation Study," Transportation Research Record No. 1035, Transportation Research Board, Washington, D.C., 1985, pp 82-84.

30. Nsour, S., et al., "Investigation of the Impacts of Ramp Metering on Traffic Flow With and Without Diversion," Transportation Research Record No. 1365, Transportation Research Board, Washington, D. C., 1992, pp. 116-124.

31. Payne, J. H., Thompson, S., and Chang, G. L., "Calibration of FRESIM to Achieve Desired Capacities" Transportation Research Board, 76th Annual Meeting, January 12-16, 1997, Paper No. 971032.

32. Pfefer, C. R., Stenzel, W. W., and Lee, D. B., " Safety Impact of the 65-mph Speed Limit: A Time Series Analysis", Transportation Research Record 1318, 1991, pp. 22 - 33.

33. Pahl, J., "Gap-Accepatance Characteristics in Freeway Traffic Flow", Transportation Research Record No. 409, Transportation Research Board, 1972, pp. 57-63.

34. Pahl, J., "Lane Change Frequencies in Freeway Traffic Flow", Transportation Research Record No. 409, Transportation Research Board, 1972, pp. 17-25.

35. Radwan, E. A. and Kalevela A. F. S., " Investigation of the Effect of Change in Vehicular Characteristics on Highway Capacity and Level of Service", Transportation Research Record 1005, Transportation Research Board, 1984, pp. 65-71.

36. Ranney, A. T. and Pulling, H. N., "Performance Differences on Driving and Laboratory Tasks Between Drivers of Different Ages" Transportation Research Record 1281, Transportation Research Board, 1990, pp. 3-9.

37. Roess, R. P. and Ulerio, J. M., "Comparison of the 1994 Highway Capacity Manual's Ramp Analysis Procedures and the FRESIM Model", NCHRP Report 385, Transportation Research Board, Washington, D.C., 1997.

38. Roess, R. P. and Ulerio, J. M., "Capacity of Ramp Freeway Junctions", NCHRP Report 3-37, Polytechnic University, Brooklyn, NY, 1993.

39. Rouphail, M. N. and Tiwari, G., "Flow Characteristics at Freeway Lane Closures", Transportation Research Record 1035, Transportation Research Board, 1985, pp. 50-58.

40. Sharma, S. C. L., et al, "Driver Population Factor in the New Highway Capacity Manual," Journal of Transportation Engineering, ASCE, Vol. 113, No.5, 1987, pp. 575-579.

41. Sharma, S. C. L., et al, "Road Classification According to Driver Population," Transportation Research Record No.1090, Transportation Research Board, 1986, pp. 61-69.

42. Sharma, S. C. L., et al, "Seasonal Traffic Counts for a Precise Estimation of AADT," ITE Journal, Vol.64, No.9, 1994, pp.21-28.

43. Skabardonis, A., Cassidy, M., May, A. D. and Cohen, S., "Application of Simulation to Evaluate the Operation of Major Freeway Weaving Sections," Transportation Research Record No. 1225, Transportation Research Board, Washington, D. C., 1989, pp. 91-98.

44. "TRAF User Reference Guide," Report No. FHWA-RD-92-103, Federal Highway Administration, U.S. Department of Transportation, 1992.

45. Vuchic, R. V., "Urban Public Transportation System and Technology", Prentice-Hall Inc., Englewood Cliffs, New Jersey 07632, 1981.

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